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CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

NOVEMBER, 1953. **Vol. XLVIII, No. 11**

FORTY-EIGHTH YEAR OF PUBLICATION

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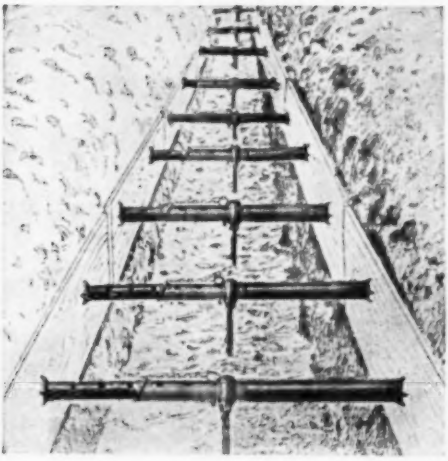

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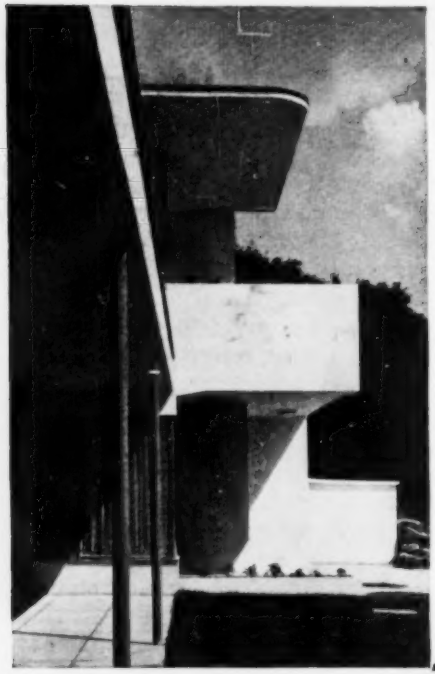

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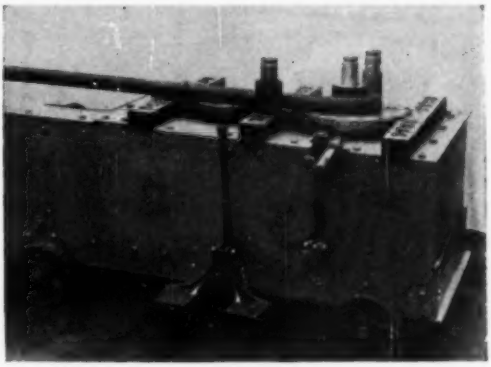

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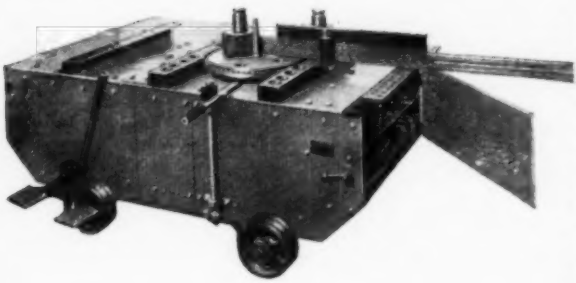

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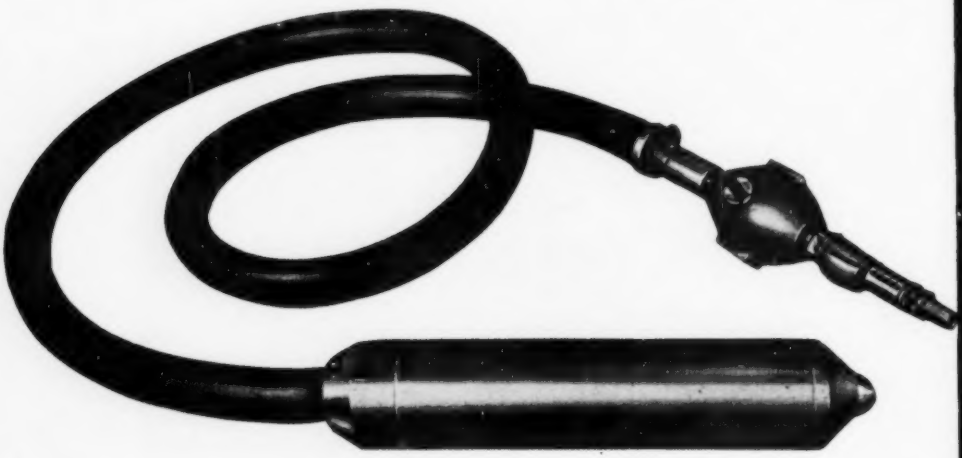

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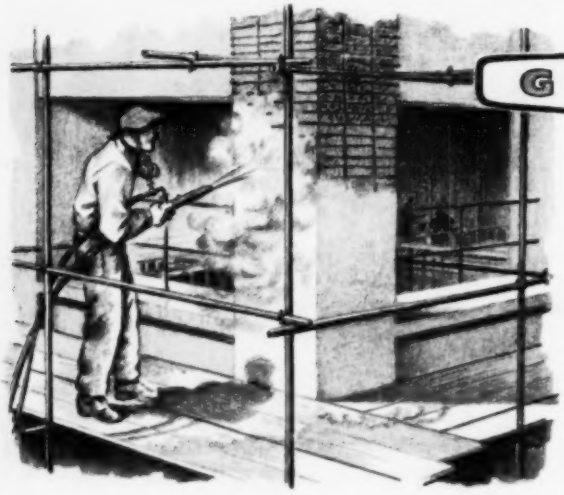

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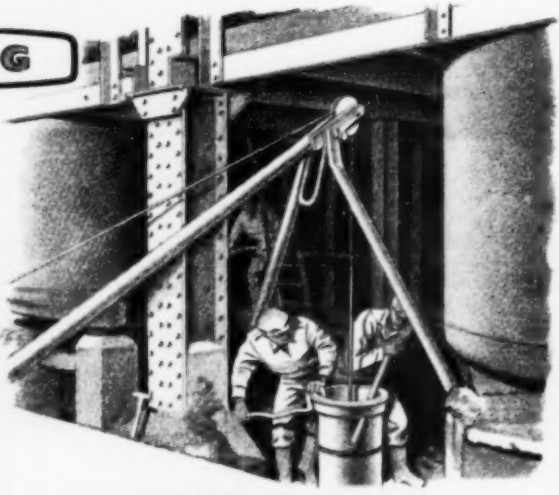

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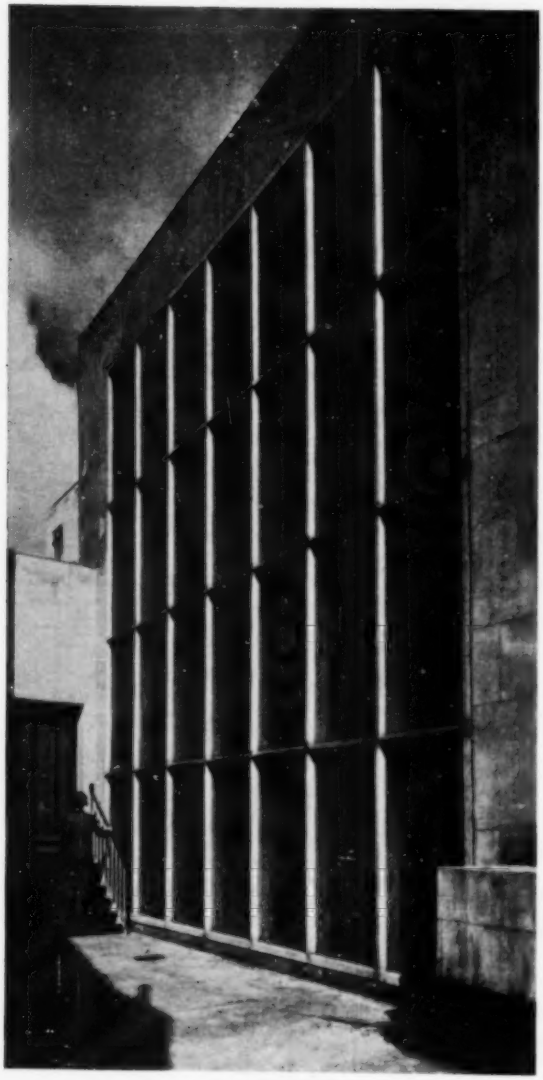

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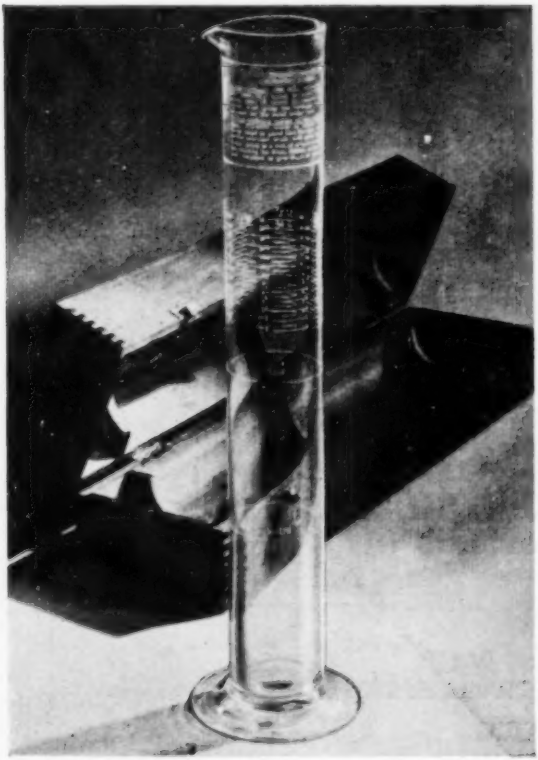

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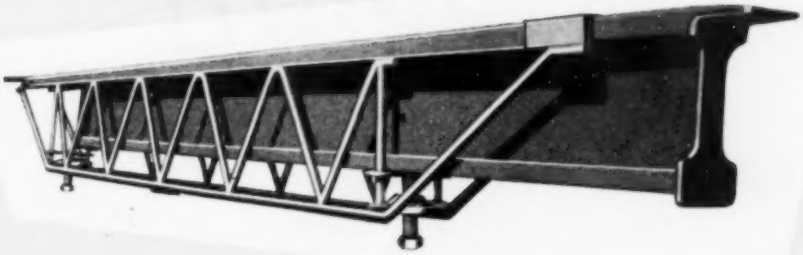

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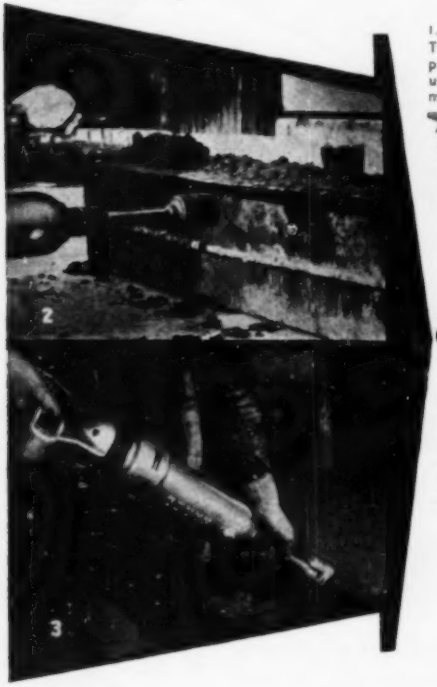

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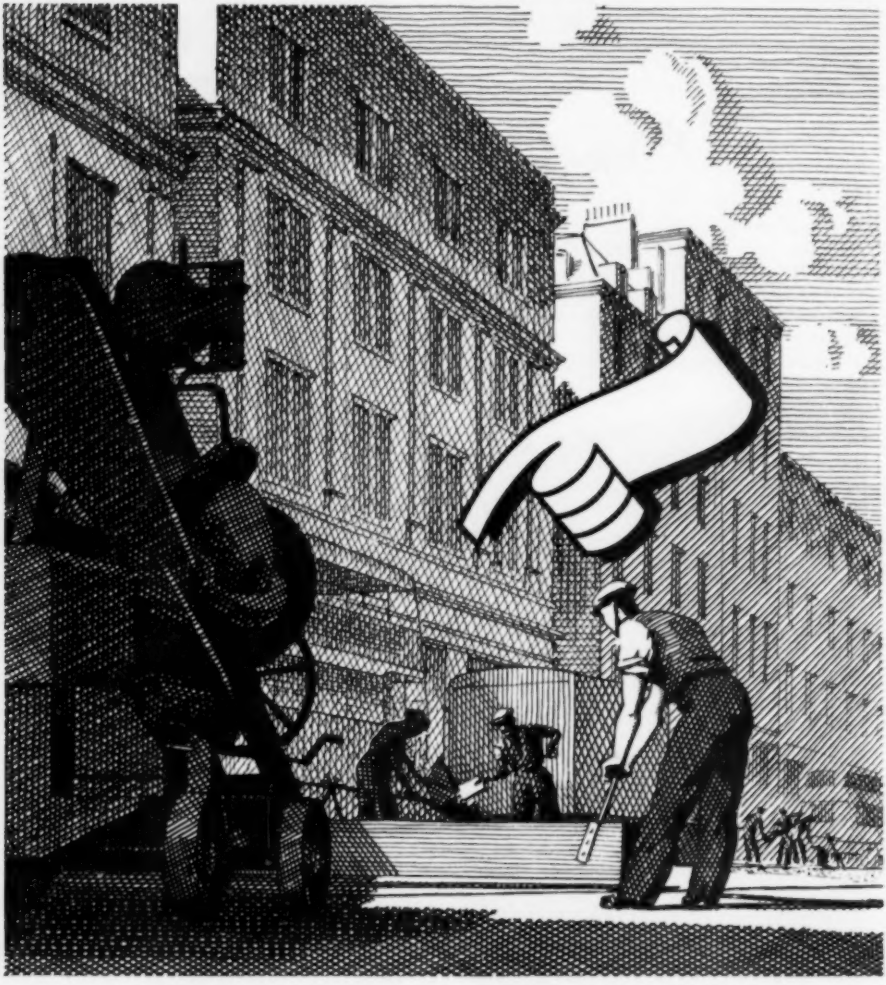

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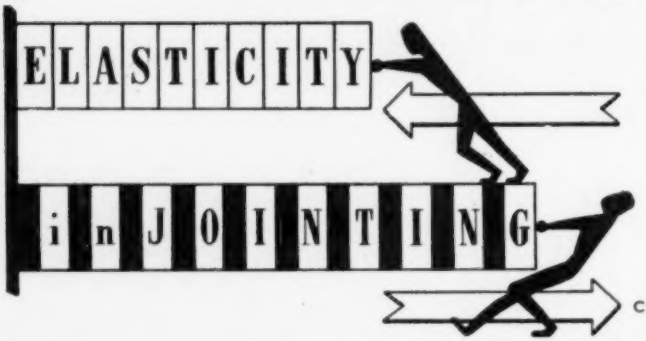

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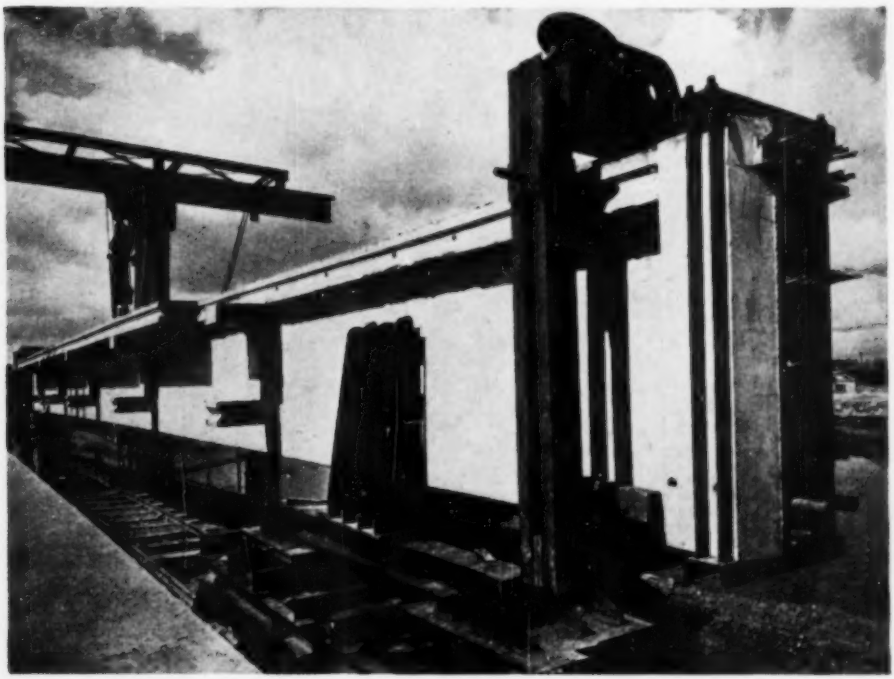

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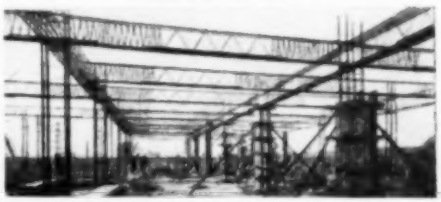

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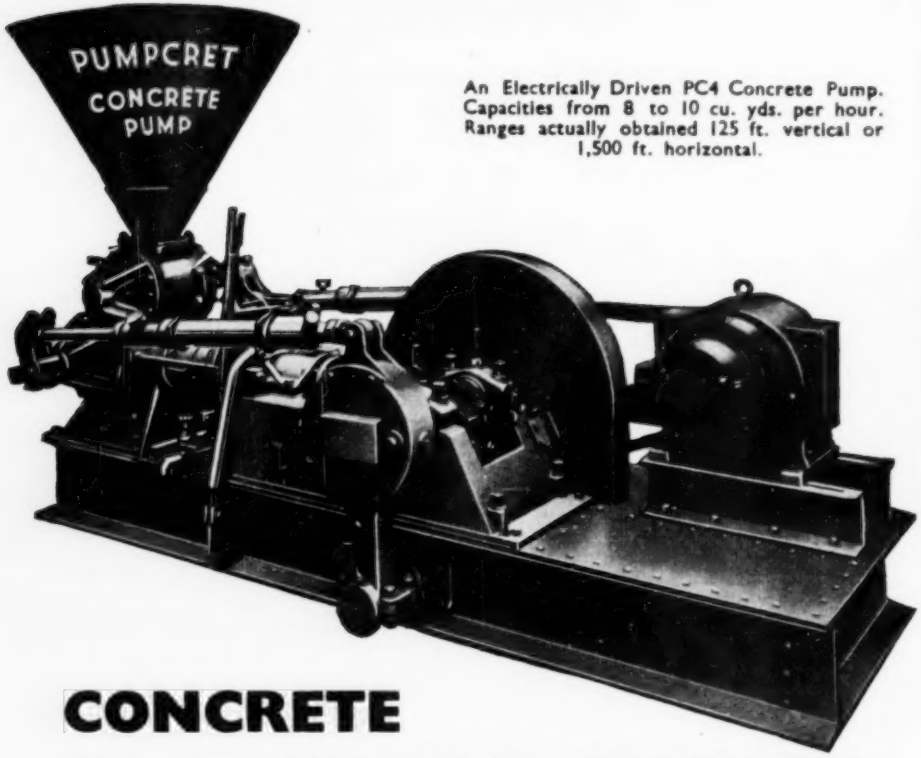

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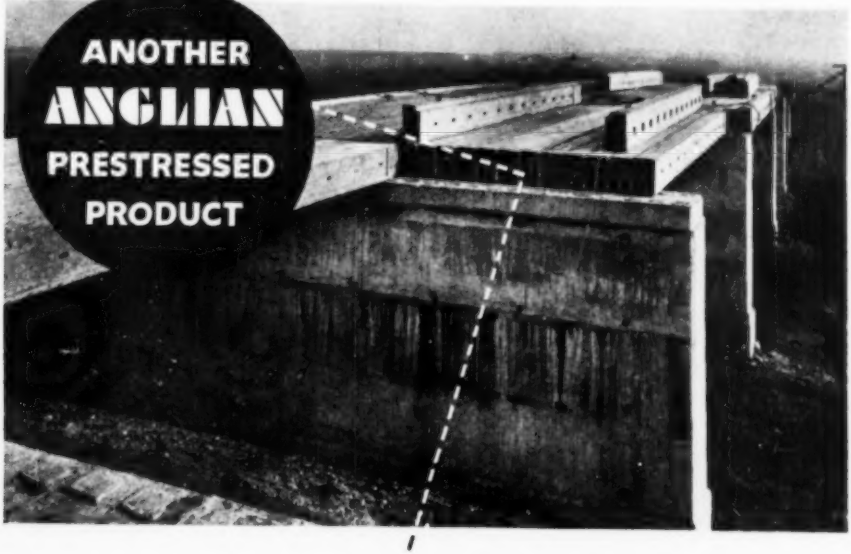

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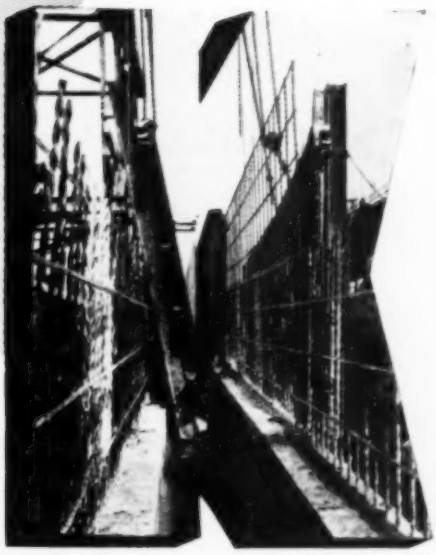

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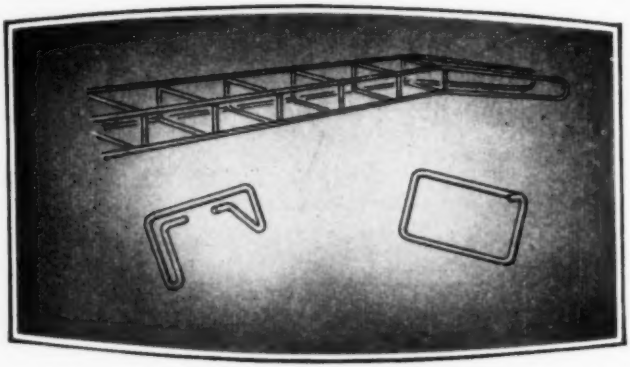

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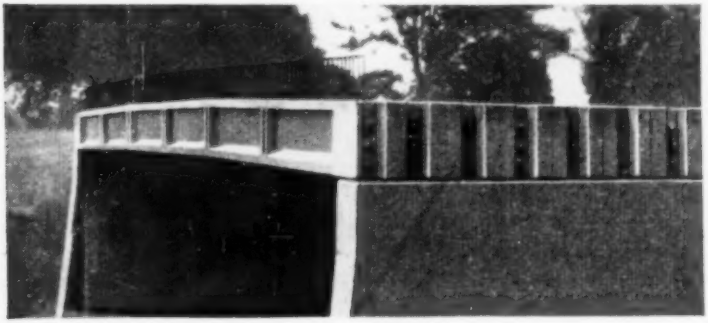

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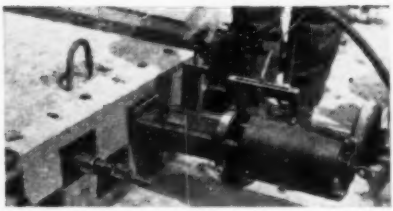

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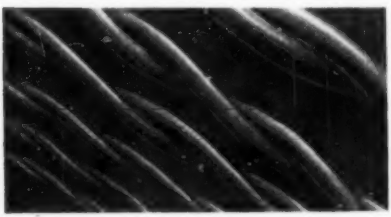

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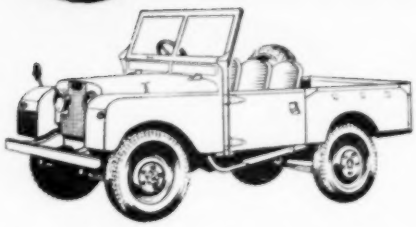

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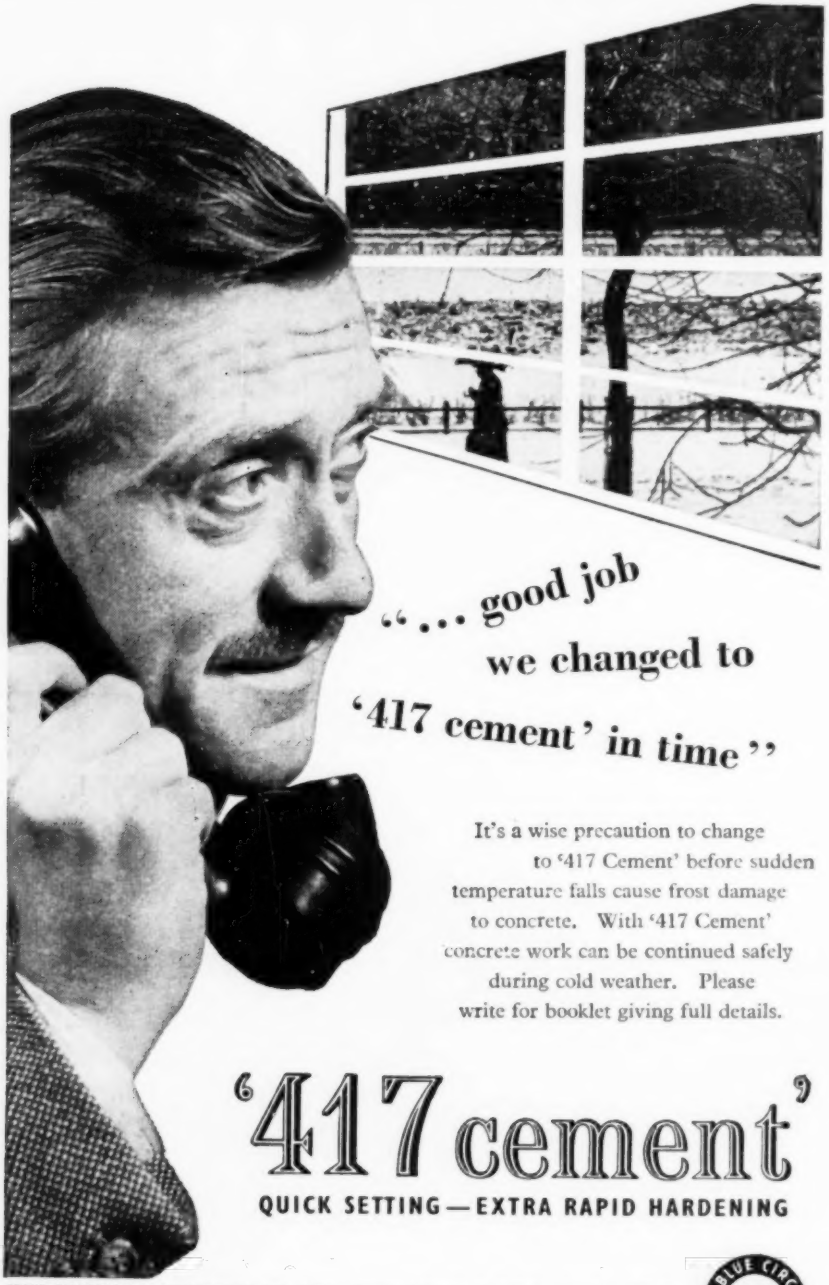

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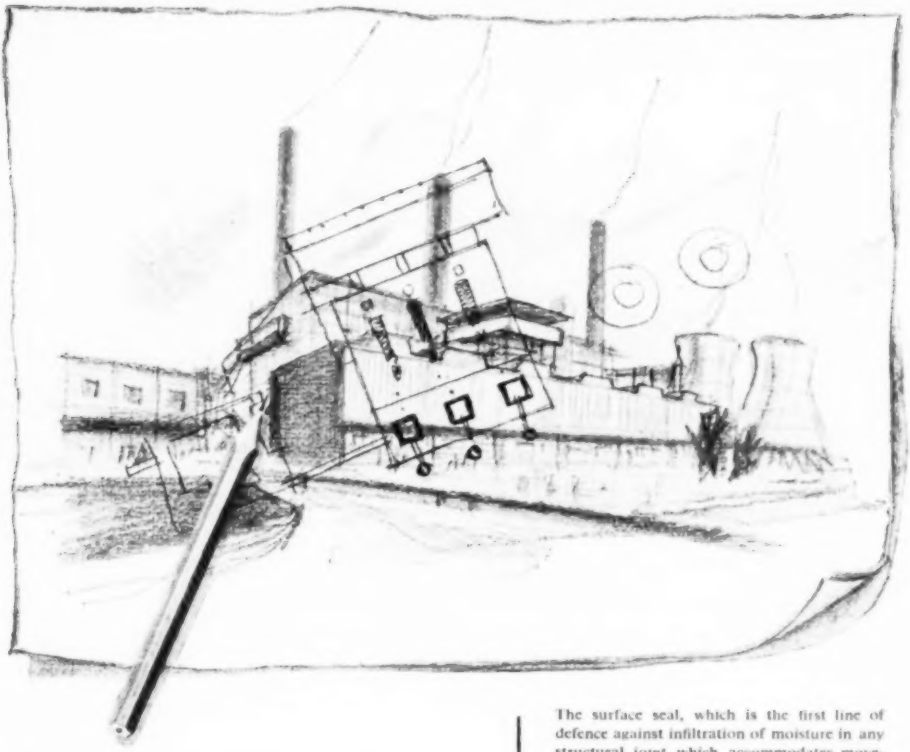

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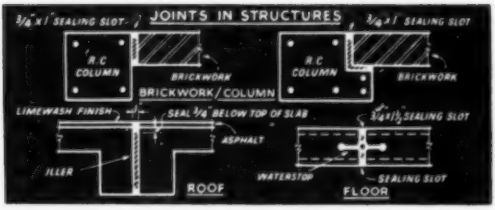

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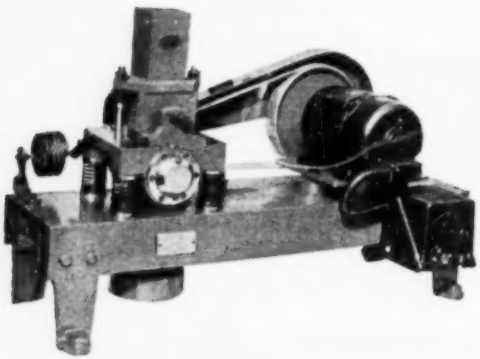

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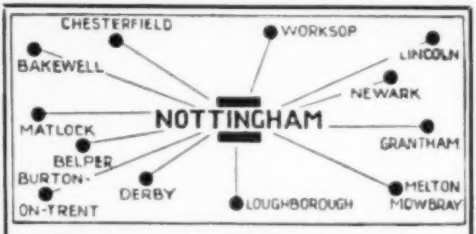

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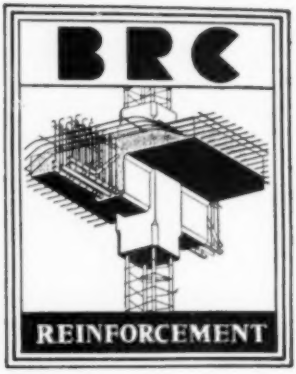

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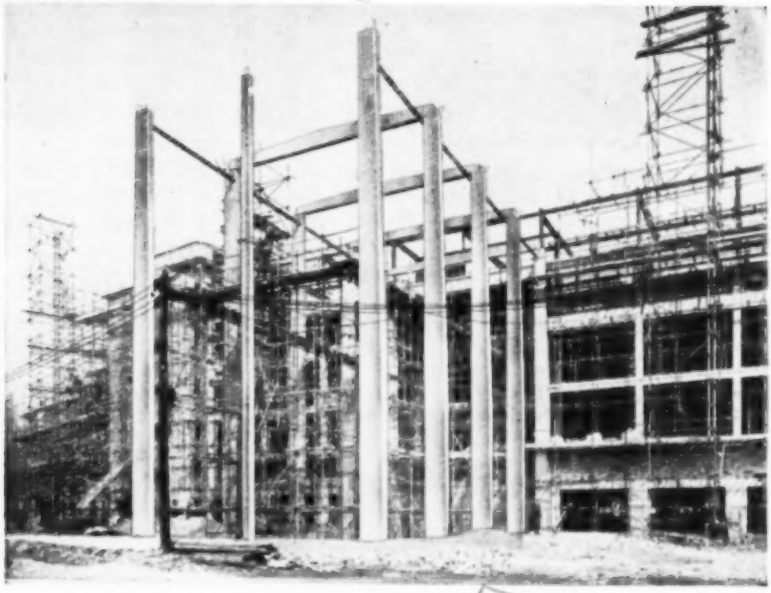

Architect: Graham Dawbarn, C.B.E., M.A., F.R.I.B.A. (Norman & Dawbarn), in association with M. T. Tudsbery, C.B.E., F.C.G.I., M.I.C.E., (The Civil Engineer to the B.B.C.) *Main Contractor:* Higgs & Hill Ltd.

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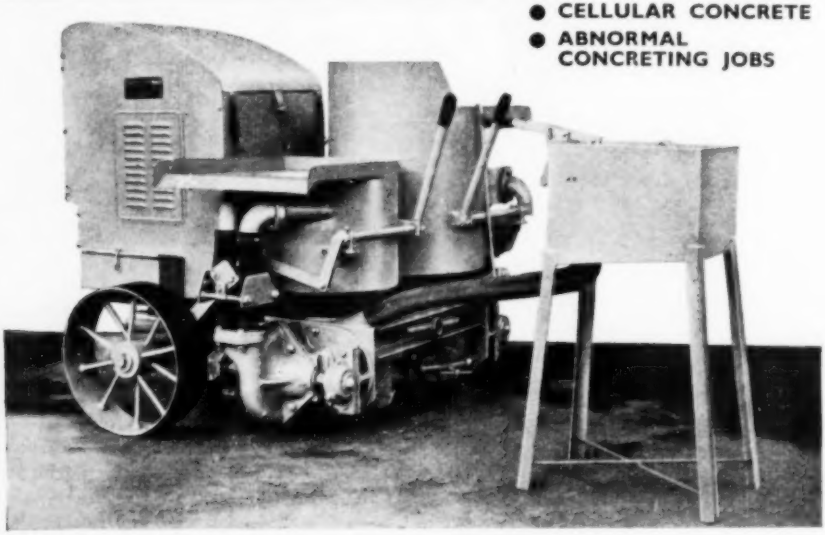

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An extract from The Times

"Producibility" is among the leading factors governing design in American engineering, according to a team of 11 British specialists who visited nearly 50 establishments in the United States early last year. Their report, *Design for Production*, published to-day by the British Productivity Council (3s. 6d.), also emphasizes the need for considering economy when deciding upon design.

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Volume XLVIII, No. 11. LONDON, NOVEMBER, 1953

EDITORIAL NOTES

Developments in Prestressed Concrete.

LAST month the International Federation of Prestressing held in London its first international congress, when papers were presented on the influence of abnormal temperatures, on design based on the loads causing failure of statically-determinate beams and slabs, and on the plastic and elastic states of statically-indeterminate structures.

Since the conference held in Ghent in 1951 and the symposium on statically-indeterminate structures held in London in the same year, prestressed concrete has become more generally accepted as a method of construction. New systems and methods of tensioning and anchoring cables, wires, and bars have been introduced and others have become more widely used. Prominent amongst the latter are the methods using large-diameter bars, which have been used for structures of all kinds and in particular in some remarkable bridges over the river Rhine and elsewhere in Germany. In the United States stranded-wire cables for applying the prestressing force appear to be used more than in Europe. Probably, however, the greatest advances have been in prestressed frames, including frames many times indeterminate for multiple-story buildings in, for example, Belgium, the Belgian Congo, and London. Slabs prestressed in two directions are also being used more frequently; an interesting Belgian example of this application is a flat-slab floor in which the thickened parts of the slab over the columns are prestressed and the remainder of the slab is of reinforced concrete. It is reported also, from the U.S.A., that slabs prestressed in two directions are being used in the method of erection in which entire floors are cast at ground level and jacked up to their final position where they are connected to steel columns by means of steel plates cast in the slabs and welded to the columns.

It is reassuring to learn that the fears expressed by some engineers regarding the fire-resistance of prestressed concrete have, to some extent at least, less justification than was anticipated. The tentative conclusions based on tests, as yet uncompleted, by the British Fire Research Station are that a fire-resistance of two hours is obtained if the cover of the concrete to the cables is $2\frac{1}{2}$ in., and that longer periods of fire-resistance may be obtained by increasing the cover of concrete or by an encasement of insulating material. It would appear important that the temperature of the high-tensile wire should be prevented from rising beyond 400 deg. C. during the period of fire-resistance required, for at temperatures greater than this there is a considerable decrease in the strength of the wire.

It is also reassuring to learn that failure of a beam due to fire is unlikely to be sudden, and that during exposure to high temperatures a progressive sagging gives warning before collapse occurs.

In the discussions of the papers presented to the congress on the load causing failure (known as the ultimate strength) of statically-determinate beams the importance was frequently emphasised of more precise definition of the conditions to which the ultimate strength is related, and it was pointed out that the greatest load which a beam can support differs according to the length of time during which the load is applied and whether the load is static or dynamic. Having regard to the nature of the materials, it can hardly be expected that the accuracy of the computed load would be within 5 to 10 per cent. of the actual load causing failure. As in the case of reinforced concrete there is considerable agreement on the load causing failure in bending of prestressed beams with small amounts of steel. There is, however, little information available regarding the strength of prestressed beams subjected to high shearing stresses, particularly when these occur where the tensile stresses due to bending are high. The lower strength that may result from the ineffective grouting of cables or bars was emphasised by many speakers ; this is a matter to which contractors and supervising engineers should give particular attention, and manufacturers of equipment for pressure grouting might consider whether the plant now available can be improved. That effective bond can be obtained when the grouting is carried out carefully is shown by a test, made on the Continent, of a beam in which the anchors at the ends of a cable were removed before the load was applied ; at the calculated working load this beam behaved as a similar beam with the anchors in place.

It is well known that statically-indeterminate structures in prestressed concrete are more difficult to design than are similar structures in reinforced concrete. Also, unless the proportions of the frames are carefully chosen there is a likelihood of such structures being less economical than statically-determinate structures. But these difficulties are being overcome ; simpler methods of design are being evolved, and the effects on continuous structures of loads near those causing failure is more clearly understood. There appears now to be a wide measure of agreement of the value, particularly at loads approaching the load that would cause failure, of untensioned mild steel or high-tensile steel in the parts of a beam subjected to tension. Tensile stresses in the concrete as high as 500 lb. per square inch have been advocated so long as sufficient untensioned steel is provided. The acceptance of such stresses makes much simpler the choice of the dimensions of a section and of the shape of the longitudinal axis of a cable or bar.

It is unfortunate that the papers prepared for the congress were not available at the meetings. Very brief summaries of the papers and comments by the three reporters were distributed to those who attended the congress, but in some cases it is difficult to know which are the views of the authors and which the views of the commentators. For this reason we are not in this journal printing the summaries. It is suggested that at the next congress the papers be available in advance, or at any rate that summaries be prepared by the authors in order that there shall be no doubt about which are the author's words and which the reporter's. A list of the papers prepared for the London congress is given on page 348.

The Calculation of Deflections and Bending Moments at the Supports of Prestressed Beams.

By E. W. BENNETT, M.Sc., A.M.I.C.E.

THE design of prestressed beams with curved cables is frequently lengthy and involved. A table of formulæ has therefore been compiled for some common profiles of cables in beams of uniform cross section, in order to simplify (a) The calculation of the initial deflection of a beam due to the prestressing force, and (b) The evaluation of the secondary moments caused by the prestressing of a statically-indeterminate structure.

Deflection due to the Prestressing Force.

According to Mohr's second moment-area theorem,

$$\left(x\frac{dy}{dx} - y\right)_0^X = \int_0^X \frac{M}{EI}\, x\, dx$$

where x is the horizontal distance from the origin, y the upward vertical deflection at any section of a beam over a length X, and M is the moment tending to produce sagging. Thus for a symmetrically-loaded beam ABA′ of length $2X$, the left-hand side of the equation represents the deflection of the mid-point B, since at this point $x = X$ and $\frac{dy}{dx} = 0$.

The internal bending moment due to a prestressing force P in a frictionless cable, at a point where the eccentricity of the cable above the centroid of the section is e, is Pe. Therefore the upward deflection due to the prestressing force is $-\frac{P}{EI}\int_0^X ex\, dx$, which may be written $\frac{X^2}{6EI}N_{BA}$ where N_{BA} is $-\frac{6P}{X^2}\int_0^X ex\, dx$

$$= -\frac{6P}{X^2}\left(\begin{matrix}\text{Moment about A of the area between the}\\ \text{cable profile and the centroidal axis}\end{matrix}\right).$$

In *Fig.* 1 the formulæ for N_{AB} and N_{BA} are tabulated for twelve profiles. The formulæ have been calculated on the assumption that the profiles are parabolic, which is sufficiently accurate for curves of the large radii necessary in practice.

The deflection at the centre of the span of a symmetrical prestressed beam may be quickly calculated by the use of the formula, based on half the span, as in the following example.

EXAMPLE 1: A simply-supported beam of 16 ft. span and 10 in. deep by 6 in. wide is prestressed with a force of 50,000 lb. by a cable of parabolic profile with an eccentricity of 3 in. below the centroid at midspan, and 2 in. above the centroid at the supports. If the modulus of elasticity of the concrete is 4,000,000 lb. per square inch, calculate the deflection of the beam due to the prestressing force.

From *Fig.* 1 (line 3), $N_{BA} = \frac{P}{2}(5e - e') = \frac{50{,}000}{2}\left[(5 \times 3) - 2\right] = 325{,}000$ in.-lb.

$$I = \frac{6 \times 10^3}{12} = 500 \text{ in.}^4 \quad X = \tfrac{1}{2} \times 16 \times 12 = 96 \text{ in.}$$

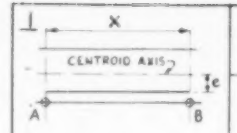
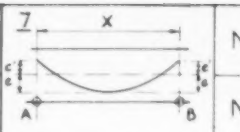
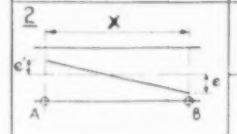
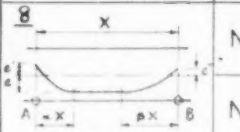
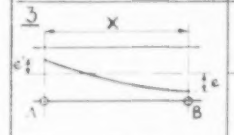
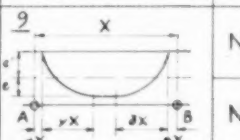
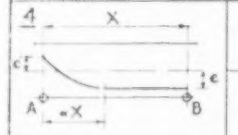
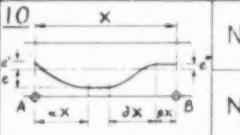
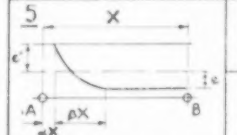
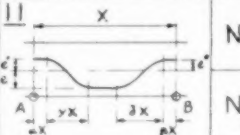
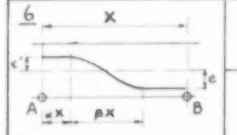
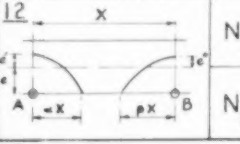
1 	N_{AB}	$+3Pe$	7 	N_{AB}	$+P\left[2e-e'\right]$
	N_{BA}	$+3Pe$		N_{BA}	$+P\left[2e-e'\right]$
2 	N_{AB}	$+P\left[e-2e'\right]$	8 	N_{AB}	$+\frac{P}{2}\left[(6+\alpha^2-\beta^2-4\alpha)e+(\alpha^2-4\alpha)e'-\beta^2e''\right]$
	N_{BA}	$+P\left[2e-e'\right]$		N_{BA}	$+\frac{P}{2}\left[(6-\alpha^2+\beta^2-4\beta)e-\alpha^2e'+(\beta^2-4\beta)e''\right]$
3 	N_{AB}	$+\frac{3P}{2}\left[e-e'\right]$	9 	N_{AB}	$+\frac{P}{2}\left[\begin{matrix}(6+6\alpha^2-6\beta^2-\gamma^2-\delta^2+4\alpha\gamma-4\beta\delta-12\alpha-4\gamma)e\\+(\gamma^2-\delta^2+4\alpha\gamma-4\beta\delta-4\gamma)e'\end{matrix}\right]$
	N_{BA}	$+\frac{P}{2}\left[5e-e'\right]$		N_{BA}	$+\frac{P}{2}\left[\begin{matrix}(6-6\alpha^2-6\beta^2-\gamma^2-\delta^2-4\alpha\gamma+4\beta\delta-12\beta-4\delta)e\\-(\gamma^2-\delta^2+4\alpha\gamma-4\beta\delta+4\delta)e'\end{matrix}\right]$
4 	N_{AB}	$+\frac{P}{2}\left[(6-4\alpha+\alpha^2)e-\alpha(4-\alpha)e'\right]$	10 	N_{AB}	$+\frac{P}{8}\left[\begin{matrix}(24+4\alpha^2-24\beta^2-72^2-24\beta\delta-16\alpha)e\\+(4\alpha^2-16\alpha)e'-(24\beta^2+72^2+24\beta\delta)e''\end{matrix}\right]$
	N_{BA}	$+\frac{P}{2}\left[(6-\alpha^2)e-\alpha^2e'\right]$		N_{BA}	$+\frac{P}{8}\left[\begin{matrix}(24-4\alpha^2+24\beta^2+72^2+24\beta\delta-48\beta-24\delta)e\\-4\alpha^2e'+(24\beta^2+72^2+24\beta\delta-48\beta-24\delta)e''\end{matrix}\right]$
5 	N_{AB}	$+\frac{P}{2}\left[(6+6\alpha^2+\beta^2-12\alpha-4\beta+4\alpha\beta)e-\beta(4-4\alpha-\beta)e'\right]$	11 	N_{AB}	$+\frac{P}{8}\left[\begin{matrix}(24+24\alpha^2-24\beta^2+7\gamma^2-72^2+24\alpha\gamma-24\beta\delta-48\alpha-24\gamma)e\\+(24\alpha^2+7\gamma^2+24\alpha\gamma-48\alpha-24\gamma)e'\\-(24\beta^2+72^2+24\beta\delta)e''\end{matrix}\right]$
	N_{BA}	$+\frac{P}{2}\left[(6-6\alpha^2-\beta^2-4\alpha\beta)e-\beta(4\alpha+\beta)e'\right]$		N_{BA}	$+\frac{P}{8}\left[\begin{matrix}(24-24\alpha^2+24\beta^2-7\gamma^2-72^2-24\alpha\gamma+24\beta\delta-48\beta-24\delta)e\\-(24\alpha^2+7\gamma^2+24\alpha\gamma)e'\\+(24\beta^2+72^2+24\beta\delta-48\beta-24\delta)e''\end{matrix}\right]$
6 	N_{AB}	$+\frac{P}{8}\left[\begin{matrix}(24+24\alpha^2+7\beta^2+24\alpha\beta-48\alpha-24\beta)e\\+(24\alpha^2+7\beta^2-24\alpha\beta-48\alpha-24\beta)e'\end{matrix}\right]$	12 	N_{AB}	$+\frac{P}{2}\left[(4\alpha-3\alpha^2+3\beta^2)e+(3\alpha^2-8\alpha)e'-3\beta^2e''\right]$
	N_{BA}	$+\frac{P}{8}\left[\begin{matrix}(24-24\alpha^2-7\beta^2-24\alpha\beta)e\\-(24\alpha^2+7\beta^2+24\alpha\beta)e'\end{matrix}\right]$		N_{BA}	$+\frac{P}{2}\left[(4\beta+3\alpha^2-3\beta^2)e-3\alpha^2e'+(3\beta^2-8\beta)e''\right]$

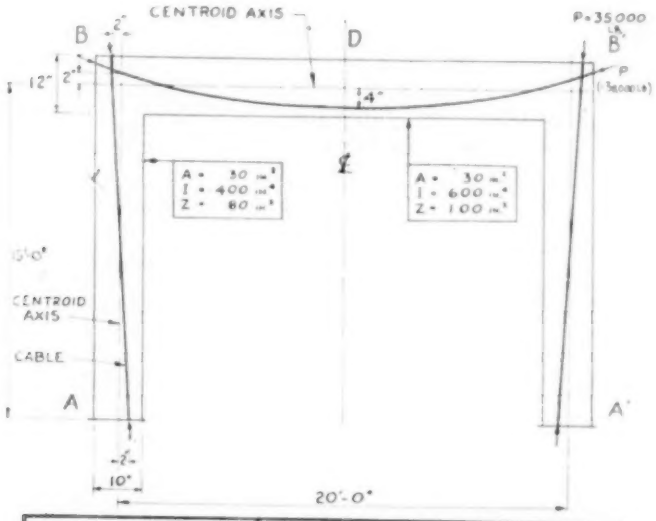
Fig. 1.

$$\text{Upward deflection} = + \frac{X^2}{6EI} N_{BA} = \frac{96^2}{6 \times 4 \times 10^6 \times 500} \times 325{,}000 = 0{\cdot}25 \text{ in.}$$

This upward deflection of $\frac{1}{4}$ in. must be subtracted from the downward deflection due to the dead and imposed loads.

Secondary Moments in Indeterminate Structures.

If an indeterminate structure is prestressed secondary moments are produced at the rigid joints, and these must be taken into account when calculating the



MEMBER	A-B				B-B			
POINT	A		B		B		D	
FACE OF FRAME I = INSIDE O = OUTSIDE	I	O	I	O	I	O	I	O
CONTINUITY MOMENT M_c (LB.-IN.)	+28,400		+13,200		-13,200		-13,200	
PRESTRESS DUE TO M_c $\pm M_c/Z$ (LB./SQUARE INCH)	-355	+355	-165	+165	-130	+130	-130	+130
DIRECT PRESTRESS (LB./SQUARE INCH)	+2040	+290	+290	+2040	+400	+1,600	+2,200	-200
RESULTANT PRESTRESS (LB./SQUARE INCH)	+1,685	+645	+125	+2205	+170	+1730	+2070	-70

Fig. 2.—A Frame with Columns Fixed at the Bases.

stresses in the concrete. They may conveniently be calculated from Clapeyron's theorem of three moments.

If M_A, M_B, and M_C are the bending moments at the ends of two continuous members AB and BC, of length L_{AB} and L_{BC} and moments of inertia I_{AB} and I_{BC} respectively, the moments are given by

$$\frac{L_{AB}}{I_{AB}} M_A + 2\left(\frac{L_{AB}}{I_{AB}} + \frac{L_{BC}}{I_{BC}}\right) M_B + \frac{L_{BC}}{I_{BC}} M_C - 6E\left(\frac{y_A - y_B}{L_{AB}} + \frac{y_C - y_B}{L_{BC}}\right)$$

$$= -\frac{6}{L_{AB} I_{AB}} \int_0^{L_{AB}} Mx \, dx - \frac{6}{L_{BC} I_{BC}} \int_0^{L_{BC}} Mx \, dx,$$

where y_A, y_B, and y_C are the upward vertical deflections of A, B, and C. The equation may be written:

$$M_A + 2(1 + k)M_B + kM_C - 6EK_{AB}(\theta_{AB} - \theta_{BC}) = N_{BA} + kN_{BC}$$

where $K_{AB} = \frac{I_{AB}}{L_{AB}}$, $k = \frac{I_{AB}L_{BC}}{I_{BC}L_{AB}}$, and θ_{AB}, θ_{BC} are the clockwise rotations of AB and BC. This enables the tabulated functions N to be introduced when it is required to calculate the moments produced by the prestressing force. In this case it should be noted that the values of N apply to the whole of the span, that is $X = L$.

EXAMPLE 2: Calculate the stresses in the concrete members of the frame shown in *Fig.* 2. As the arrangement of the cables is symmetrical the frame is twice indeterminate with respect to the prestressing system, and the moments M_A and M_B at the supports of members AB and for BB′ may be obtained from the theorem of three moments, θ being zero throughout.

For AB: $$2M_A + M_B = N_{AB} \quad . \quad . \quad . \quad . \quad . \quad (1)$$

For ABB′: $$M_A + (2 + 3k)M_B = N_{BA} + kN_{BB'} \quad . \quad . \quad . \quad (2)$$

Solving equations (1) and (2):

$$M_A = \frac{(2 + 3k)N_{AB} - N_{BA} - kN_{BB'}}{3(1 + 2k)}, \quad M_B = \frac{2N_{BA} - N_{AB} + 2kN_{BB'}}{3(1 + 2k)}.$$

From *Fig.* 1 (line 2):

$$N_{AB} = P(e - 2e') = 35{,}000[-2 - 2(-2)] = +70{,}000 \text{ in.-lb.}$$
$$N_{BA} = P(2e - e') = 35{,}000[2(-2) - (-2)] = -70{,}000 \text{ in.-lb.}$$

From *Fig.* 1 (line 7):

$$N_{BB'} = P(2e - e') = 30{,}000[(2 \times 4) - 2] = +180{,}000 \text{ in.-lb.}$$

$$k = \frac{400}{600} \times \frac{20}{15} = 0{\cdot}89.$$

$$M_A = \frac{(4{\cdot}67 \times 70{,}000) - (-70{,}000) - (0{\cdot}89 \times 180{,}000)}{8{\cdot}33} = +28{,}400 \text{ in.-lb.}$$

$$M_B = \frac{2(-70{,}000) - (70{,}000) + (1{\cdot}78 \times 180{,}000)}{8{\cdot}33} = +13{,}200 \text{ in.-lb.}$$

The stresses due to the secondary bending moments caused by prestressing can now be calculated for any part of the frame. These should be added to the primary stresses due to prestressing, obtained by the usual methods. The calculation may conveniently be tabulated as in *Fig.* 2. The stresses due to the bending moments when the frame is loaded are added to the final values of the stresses due to prestressing. It should be noted that the weight of the member BB′ is always present, so that the stresses in the frame will be less than those due to prestressing alone.

If it is preferred to use the moment-distribution method for the calculation of the moments at the supports, the fixed-end moments M_{FA} and M_{FB} for the profiles of the cables shown in *Fig.* 1 are:

$$M_{FA} = \frac{2N_{AB} - N_{BA}}{3}, \text{ and } M_{FB} = \frac{2N_{BA} - N_{AB}}{3}.$$

By the use of the equations previously derived from the formulæ given in *Fig.* 1, a method may be evolved for the design (as opposed to the analysis, already described) of indeterminate prestressed structures. The following example illustrates the application of this method to a simple case.

EXAMPLE 3. Three prestressed precast beams 1 ft. 8 in. deep and of uniform symmetrical cross section are to be assembled into a continuous beam over four supports, as in *Fig.* 3, by means of parabolic " cap-cables " over the two inside supports. The cross-sectional area of the concrete is 100 sq. in. and the moment of inertia 4000 in.^{4}. Design a suitable cap-cable to produce stresses at the supports of 1500 lb. per square inch at the top and 100 lb. per square inch at the bottom of the beam.

Let the cap-cables be anchored at a distance of z ft. from the inside supports,

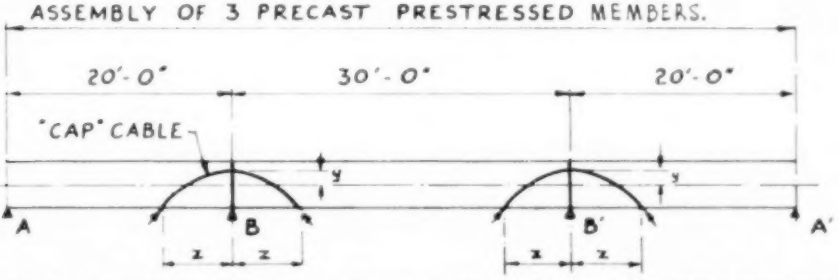


Fig. 3.—Elevation of Continuous Beam (cables in Precast Members Omitted).

and let the eccentricity of each cable at these supports be y in. The stress required at the centroid of the section is $\frac{1500 + 100}{2} = 800$ lb. per square inch. Therefore the force P required in the cap-cable is $800 \times 100 = 80{,}000$ lb. and the total bending moment required is

$$\frac{\text{(Final stress} - \text{stress at the centroid)} \times \text{moment of inertia}}{\text{Distance from the centroid to the point at which the stress is required}} = \frac{(1500 - 800)4000}{10} = 280{,}000 \text{ in.-lb.}$$

Secondary moment $M_B = \frac{N_{BA} + kN_{BB'}}{2 + 3k}$, and from *Fig.* 1 (line 12)

$$N_{BA} = \frac{80{,}000}{2}\left\{\left[4 \times \frac{z}{20} + 0 - 3\left(\frac{z}{20}\right)^2\right]10 - 0 + \left[3\left(\frac{z}{20}\right)^2 - 8 \times \frac{z}{20}\right]y\right\}$$
$$= (80{,}000 - 16{,}000y)z - (3000 - 300y)z^2.$$

$$N_{BB'} = \frac{80{,}000}{2}\left\{\left[4 \times \frac{z}{30} - 3\left(\frac{z}{30}\right)^2 + 3\left(\frac{z}{30}\right)^2\right]10 + \left[3\left(\frac{z}{30}\right)^2 - 8 \times \frac{z}{30}\right]y - 3\left(\frac{z}{30}\right)^2 y\right\} = (53{,}333 - 10{,}667y)z.$$

$$k = \frac{I}{20} \times \frac{30}{I} = 1{\cdot}50.$$

$$M_B = \frac{(80{,}000 - 16{,}000y)z - (3{,}000 - 300y)z^2 + 1{\cdot}50(53{,}333 - 10{,}667y)z}{2 + (3 \times 1{\cdot}50)}$$
$$= 24{,}600z - 4920yz - (462 - 46{\cdot}2y)z^2.$$

Total moment = Secondary moment + Direct moment of prestressing force $= M_B + Py$; therefore

$$280{,}000 = 24{,}600z - 4920yz - (462 - 46{\cdot}2y)z^2 + 80{,}000y, \text{ and}$$
$$y = \frac{462z^2 - 24{,}600z + 280{,}000}{80{,}000 - 4920z + 46{\cdot}2z^2}.$$

Any values of y and z which satisfy this relationship will produce the required stresses. These dimensions may therefore be selected with regard to practical considerations such as the desirability that the curvature of the cable should not exceed a certain value. Obtaining the values of y for assumed values of z, and calculating the corresponding radii of curvature R, we have:

$z = 5$ ft.	$y = 3{\cdot}0$ in.	$R = 11{\cdot}5$ ft.
$z = 6$ ft.	$y = 2{\cdot}9$ in.	$R = 16{\cdot}7$ ft.
$z = 7$ ft.	$y = 2{\cdot}7$ in.	$R = 23{\cdot}0$ ft.

If, for example, it is desired to limit the radius of curvature to 20 ft., it will be seen by interpolation that z should be about 6 ft. 6 in. and y about $2\frac{3}{4}$ in.

It is necessary to ensure that the stresses due to prestressing are within the required limits at other points along the beams, but generally, if the form of cable profile has been carefully selected with regard to the bending-moment diagram, this should not lead to any difficulty.

This method may also be used for dimensioning the chosen profile of the cable to give a "concordant cable" and thus zero moments due to prestressing.

Congress on Prestressed Concrete.

THE first congress of the International Federation of Prestressing was held in London on 6 to 9 October last. M. Freyssinet was elected president, but in his absence Professor Magnel presided at the congress. The next congress, the date of which has not yet been decided, is to be held in Holland.

Papers prepared for the congress were on three subjects, namely, the influence of abnormal temperatures, design based on the loads causing failure (the "ultimate" load) of statically-determinate beams and slabs, and statically-indeterminate structures in the plastic and elastic states. The general report on the first topic, presented by Mr. A. W. Hill, was based on the work of the British Fire Research Station and a paper by Mr. G. Huyghe on the effect of low temperatures. The chairman at this meeting was Mr. Ulf Bjüggren. Papers on design based on ultimate loads were prepared by Dr. P. W. Abeles, Mr. A. S. G. Bruggeling, Professor G. Magnel, Dr. P. Moenaert, and M. U. Smet; the reporter was Professor Magnel and the chairman was Professor E. Torroja. Statically-indeterminate structures were discussed under the chairmanship of Professor F. Levi and the general report was by M. Y. Guyon. Papers on this subject were prepared by Mr. A. Amirikian, M. Guyon, Professor A. M. Haas, M. R. de Keyser, M. P. Lebelle, Professor F. Levi, Professor G. Magnel, M. A. Paduart, Professor T. A. Sarmento, M. G. A. Steinman, Dr. F. G. Thomas, Bureau Socol, Schokbeton and Spoorwegbouwbedruf. We understand that the papers and discussions will later be published in book form.

[The subjects discussed at the congress are referred to in the Editorial Notes of this number.]

A Grandstand in South Africa.

A GRANDSTAND (*Figs.* 1 and 2) being constructed at Newlands, near Cape Town, South Africa, has a total seating capacity of 9000. The total length of the stand is 490 ft., comprising seven sections of 70 ft. As the width between the road and the playing field is small, the design shown in *Figs.* 3 and 4 was evolved.

The stand comprises reinforced concrete frames at 17-ft. 6-in. centres, with the seating-beams and floors spanning between the frames. The roof is supported by cantilever beams with spans of 51 ft. 6 in. The upper ramp accommodates ten rows of seats, and a continuation of the upper ramp, cantilevering 32 ft., has seven rows of seats. Under the upper ramp are committee rooms and refreshment rooms. The lower ramp accommodates fourteen rows of seats, under which is a tea-room extending the whole length of the stand. Dressing rooms are provided in the basement. *Fig.* 3 is a section through a typical frame, and *Fig.* 4 is a section through the special frame at the stairs.

The floors are designed for an imposed load of 100 lb. per square foot. The imposed load on the roof is 15 lb. per square foot, as it is not necessary in Cape Town to allow for any load due to snow. The frame was calculated as nine times

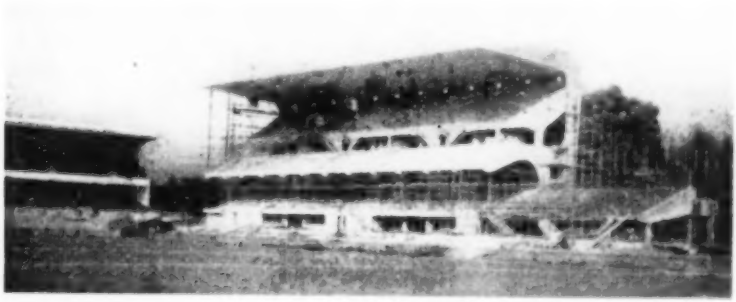


Fig. 1.—Part of the Grandstand.

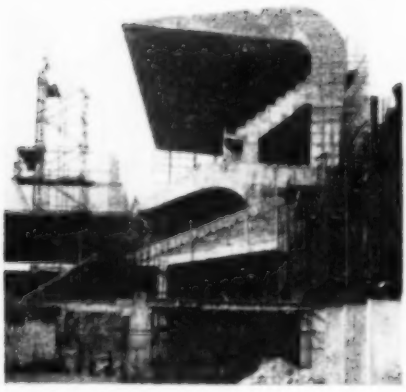


Fig. 2.—A Frame at a Joint.

indeterminate (*Fig.* 5), separated by a permanent hinge at point 11 from the rest of the structure. As it was later required that beam No. 10–11 should be enlarged as shown by the hatched part of the beam on the larger-scale cross section, the influence of the bending moment at point 10 was taken into account separately later.

The bending moments X_1 to X_9 for the statically-determinate case shown in *Fig.* 5 were assumed as the unknown quantities and were calculated by the method of plastic design explained in "Design of Indeterminate Structures by the 'Plastic' Method", by Dr. R. Gartner, in "Concrete and Constructional Engineering" for January and February, 1953. As there was no reason to adjust the corner moments, the residual values were kept quite small. The maximum

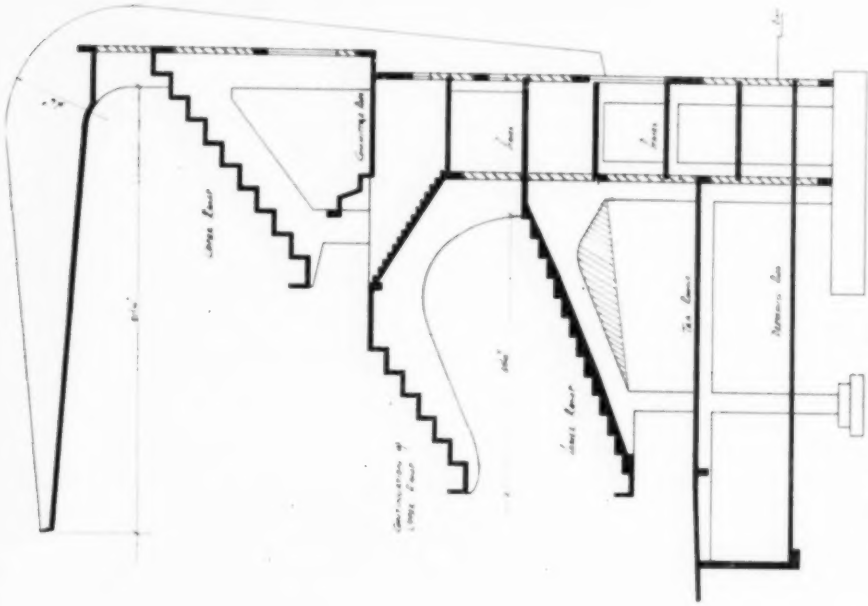


Fig. 4.—Section through a Frame at the Stairs.

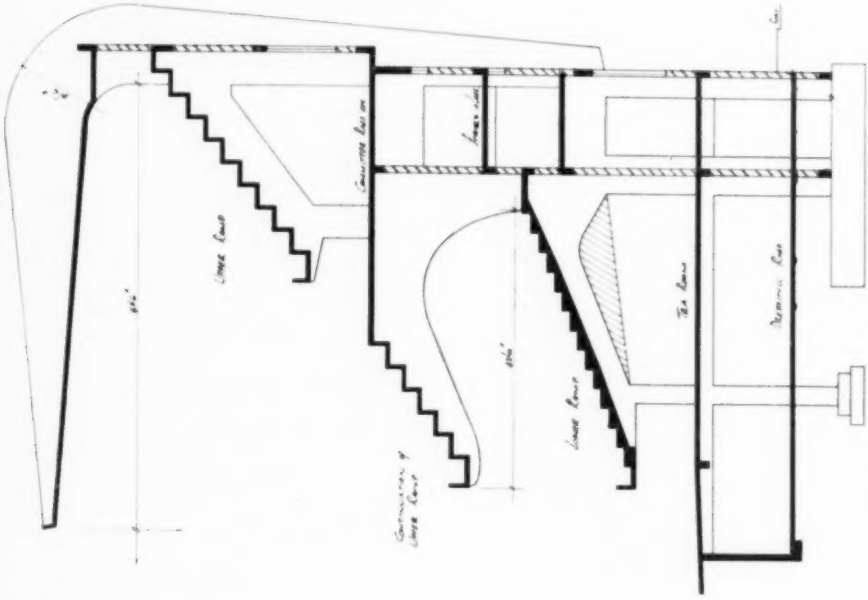


Fig. 3.—Section through a Typical Frame.

and minimum bending moments were calculated for the different positions of the loads; for greatest load the centre of gravity of the structure coincides with the centre of column 7–8. The effects of an increase and a decrease of temperature of 30 deg. F. were taken into account.

The load due to wind was assumed to be 15 lb. per square foot. The bending moments due to the wind were neglected for the upper part of the structure as they are small compared with the vertical loads, but they were included in the calculations for the lower part of the structure.

In each frame the greatest bending moment of 33,240,000 in.-lb. is on the cantilevered roof beam. The dimensions of this beam at the junction with the column are 9 ft. 6 in. by 1 ft. 6 in., reduced towards the end to 1 ft. 2 in. by 9 in. The reinforcement in each beam comprises seventeen 1¼-in. bars, which were welded to avoid congestion. The bending moment on the cantilever beam of the ramp is 30,000,000 in.-lb. The dimensions of the beam were chosen for reasons of appearance, but the adjoining member, which is subjected to a bending moment of 27,500,000 in.-lb. and a direct tensile force of 176,000 lb., is only 6 ft. by 1 ft. 6 in. These beams are reinforced with 1¼-in. bars, of which there are 26 in the top and 10 in the bottom. The shearing stress is 184 lb. per square inch and is nearly constant over the whole of the beam. The shearing force is resisted entirely by stirrups and bent-up bars. Column 1–2 is 4 ft. by 1 ft. 6 in. and the shearing force, producing a constant stress of 150 lb. per square inch, is resisted by stirrups and bent-up bars.

The concrete mixture for the frames is 1 : 1 : 2 and for the remainder of the structure 1 : 2 : 4. The concrete in the frames was vibrated. The stresses are in accordance with British Standard Code of Practice No. 114.

The expansion joints between the sections 70 ft. long were formed by fibre-board and copper strips. The roof slab is covered with sheets of malthoid, and the expansion joints between beams are covered with copper copings. The whole of the shuttering was of steel sheets supported by steel scaffolding. To facilitate propping of the roof the upper part of the upper ramp was not concreted until the shuttering for the roof was stripped. Only the upper steps were concreted with the frame, as this was necessary to prevent buckling. The centering was removed from the free end first. Unfortunately the deflection is not known as the reference marks had been moved. As the brick walls at the back of the structure are a counterweight for the cantilevers, these walls were built before the shuttering of the cantilevers was stripped.

The consulting engineers were Mr. A. F. Bisschop, M.I.Struct.E., A.M.(S.A.)I.C.E., and Dr. R. Gartner, M.I.Struct.E. The general contractors were Stewart Arnott Construction Co. (Pty.), Ltd.

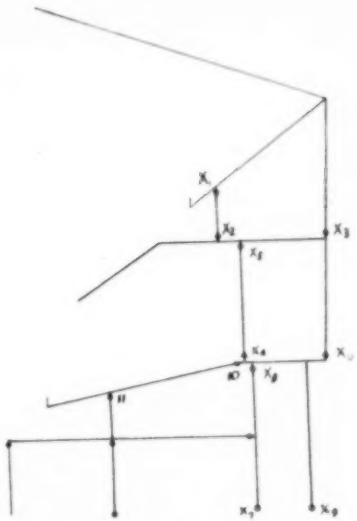


Fig. 5.—Statically-determinate Case.

Symposium on Concrete.

The Cement and Concrete Association informs us that the symposium on the quality of concrete (mentioned in our October number) has been postponed to May 11–13, 1954.

A Wind-shield Wall.

THE wall shown in *Fig. 2* surrounds a settling tank of 4,500,000-gallons capacity at the Palliser Works, Hartlepool, of the Steetley Magnesite Co., Ltd. The tank has a diameter of 240 ft. and the wall extends along the outer edge of a footpath, about 3 ft. wide, around the perimeter of the tank. The object of the wind-shield is to prevent disturbance due to wind to the settling of the precipitate in the tank.

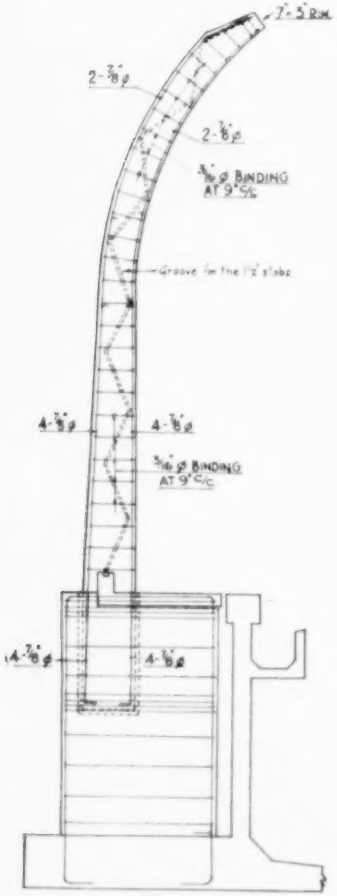


Fig. 1.—Details of Precast Columns.

The wall comprises precast curved columns extending about 19 ft. above the footpath at about 13 ft. centres. Between the columns are corrugated slabs 1½ in. thick. The wall is shown during construction in *Fig. 3*. The columns (*Fig. 1*) have a cross section at the bottom of 1 ft. 9 in. by 9 in. and were grouted in holes 2 ft. 0½ in. by 1 ft. by 4 ft. 1½ in. deep, formed in concrete bases 5 ft. 3 in. by 2 ft. by 8 ft. 3 in. deep. For about half of the perimeter of the tank the bases are on a continuous footing 1 ft. 9 in. thick and extending 7 ft. 8 in. from the inner face of the wall of the tank. The remainder of the bases are on separate footings 6 ft. 6 in. wide by 4 ft. 9 in. by 1 ft. 9 in. thick. Grooves, 1 in. deep, were formed on two faces of the columns to receive sheets of expanded metal bent to a corrugated shape. The expanded metal is wired to a ⅜-in. bar extending between the columns and lapped with ⅜-in. dowels projecting from the columns. The slab was formed by plastering both sides of the expanded metal with cement and sand mortar to a thickness of 1½ in. The corners of the slab were supported by steel angles to prevent their sagging while the mortar was hardening. The purpose of the corrugated shape of the slabs is to increase their resistance to bending caused by the wind, the force of which was assumed to be 15 lb. per square foot.

The first settling tank, constructed at these works before the last war, had a timber wind-shield but, due to the restriction on the use of timber, the British Periclase Co., Ltd. (an associated company of the Steetley Magnesite Co., Ltd.) proposed the use of a corrugated concrete wall. The details of the reinforced concrete work were prepared by the Indented Bar & Concrete Engineering Co., Ltd., who also supplied the mild steel reinforcement. The expanded metal was supplied by the Expanded Metal Co., Ltd., and the work was carried out by Messrs. Costain-John Brown, Ltd., under the direction of Mr. P. J. Metcalf, A.M.Inst.C.E., engineer of the British Periclase Co., Ltd.

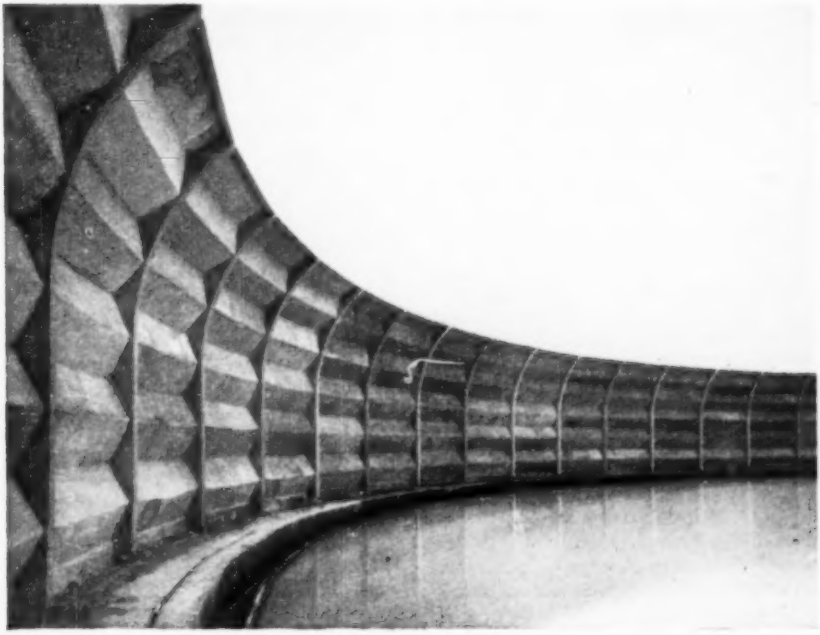


Fig. 2.—Part of Completed Wall.

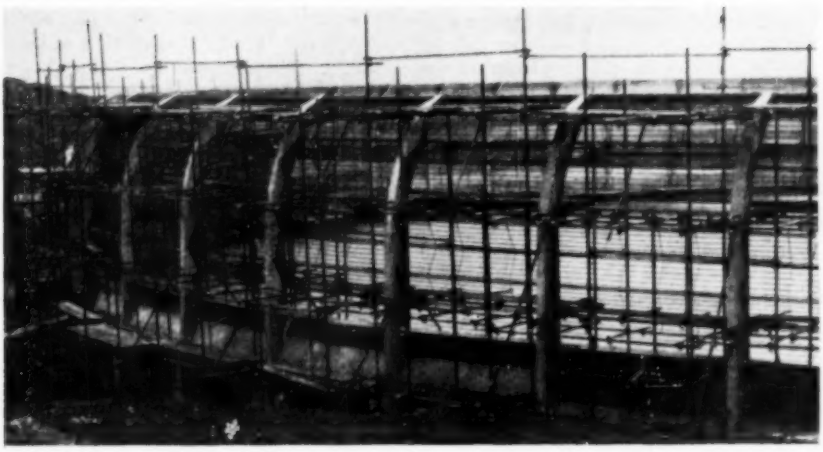


Fig. 3.—Wall During Construction.

Book Reviews.

"Theory and Practice of Structural Design applied to Reinforced Concrete." By B. Eriksen. 402 pages. (London: Concrete Publications, Ltd. 1953. Price 25s.; 5.50 dollars in North America.)

THIS comprehensive book has as its object the inclusion in one volume of all the information necessary to enable a student to progress from consideration of the elementary laws of statics to the analysis of highly-redundant indeterminate structures. The principles demonstrated apply equally to all elastic materials and materials which for the purposes of design are presumed to act in accordance with Hooke's law. Examples of the application of the theory are directed mainly to reinforced concrete, and sufficient is given of the theory of reinforced concrete to enable structures of this material to be designed.

The book is divided into two parts of which the first deals with fundamental principles and statically-determinate structures. Starting with the composition and resolution of forces, graphic statics, the concept of bending moments, and the geometrical properties of structural members, the author then deals with the calculation of internal forces and stresses and the resulting deformation and strain. Methods of calculating the slope and deflection of a beam by the integration of general expressions and by Mohr's theorem are given. A chapter on simple beams and cantilevers deals with the calculation of bending moments and shearing forces on such members and is particularly clear in the explanation of the construction of influence lines and the calculation of bending moments due to moving loads.

The second part of the book deals with the analysis of statically indeterminate structures. For structures in which sway need not be considered the author's opinion is that the method of fixed points is the most suitable and this method is explained in detail. The theorem of three moments, the theory of slope-deflection, the method of virtual work, and that due to Müller-Breslau in which the " elastic centre " of a frame is utilised are described and applied to single-bay frames, multiple-bay frames in multiple-story structures, trestles, and Vierendeel girders. Beams with variable moment of inertia are dealt with in a separate chapter.

There are many numerical examples, and an appendix contains complete calculations for the design of a portal frame bridge with beams of varying depth and supported on piled foundations.

"Beton-Kalendar." 1953. (Berlin: Wilhelm Ernst & Sohn. 1953. Price 16 D.M.)

THIS edition (in the German language) includes additional chapters on the theory of thin slabs and the design of staircases, and summaries of the Belgian and Brazilian codes of practice. The sections on construction and site practice, walls in buildings, the design of solid floor slabs, and tubular scaffolding have been partially revised. The latest recommendations of the German codes of practice are included, but no rules have yet been issued for the design of prestressed concrete to which the only reference is a description of methods used in the construction of bridges. Consideration is being given in Germany to a simplified method for the design of reinforced concrete and the preliminary recommendations are given. The feature in the chapter on the design of beams and frames is the vast amount of data which will be found sufficient for the design of any likely arrangement of beams or frames and combinations of loads. The small type makes the book difficult to read.

"Building Construction and Drawing." Part I. Elementary Course. By George A. and A. M. Mitchell. (London: B. T. Batsford, Ltd. 1953. Price 15s.)

THIS is the twenty-first edition of a book that is widely recognised as a standard textbook for students of building construction. It has again been revised and the relevant parts have been altered to agree with the By-laws of the London County Council (1952) and the Model Building By-laws (1952) of the Ministry of Housing and Local Government.

"Strength, Safety, and Economical Dimensions of Structures." By A. I. Johnson. (Stockholm: Royal Institute of Technology. 1953. No price stated.)

THIS is a statistical study of the effects on the strength of a structure or a structural member of variations in the strengths of materials, various arrangements of loads, shapes of the cross sections of members, and probable maximum loads. In the last chapter these factors are related to the design of structures having regard to the cost of the failure of a structure.

A Method of Designing Slabs.

THE following description of a method of designing slabs is based upon notes by Mr. J. Bar, of Haifa, Israel, who informs us that he has designed many buildings in which such slabs have been used during recent years in Israel.

The design is based upon the deflections of a slab freely supported on columns and without beams. Experiments show that the deformation is influenced by the amount, position, and direction of the reinforcement as well as by the spacing of the columns and the dead and imposed loads. The general equation expressing the deflections of a slab is given by Professor Timoshenko in " Theory of Plates and Shells " in the form

$$\frac{\delta^4 w}{\delta x^4} + 2\frac{\delta^4 w}{\delta x^2 \delta y^2} + \frac{\delta^4 w}{\delta y^4} = \frac{q}{D}$$

in which x and y are the rectangular coordinates of the point considered, w is the displacement of the point, q the intensity of the distributed load, and D the flexural rigidity of the slab, that is

$$D = \frac{Eh^3}{12(1 - \nu^2)}$$

where E is the modulus of elasticity, h the depth of the slab and ν is Poisson's ratio. In this equation the effect of the bars is limited to their influence on the moment of inertia of the slab, and no account is taken of the effect of their shape and direction. By measuring the deflections of slabs of the same size and with the same spans and loads, but with the bars arranged in different ways, it was possible to obtain various values for the

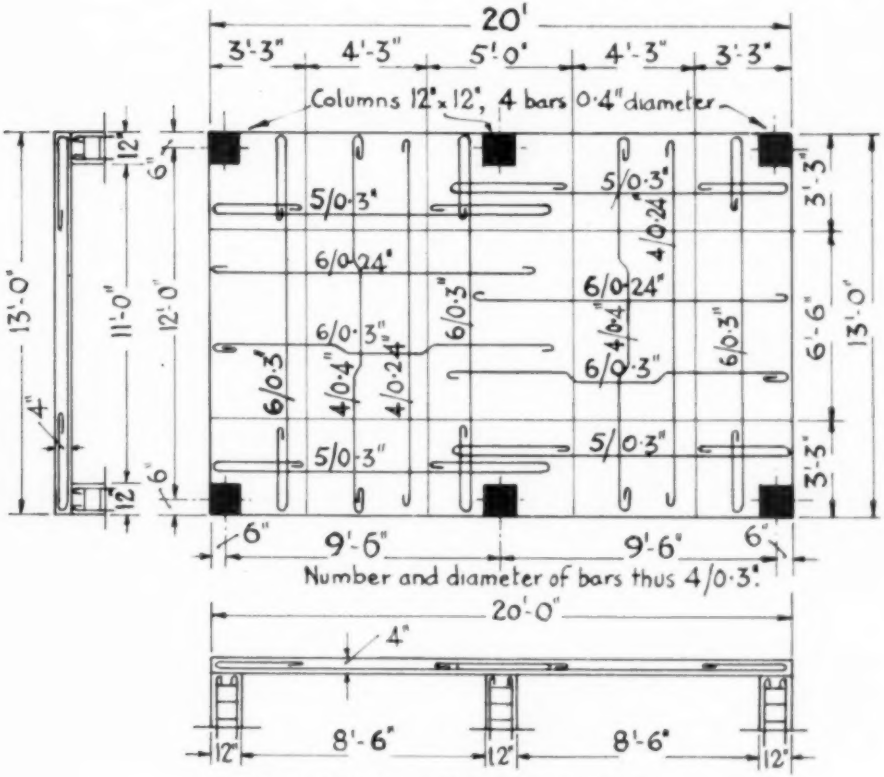


Fig. 1.—Details of Slab 20 ft. by 13 ft.

maximum stresses and positions of the greatest bending moments.

The empirical method used as a result of the tests is therefore based on the proposition that the shape of the deflected surface of a reinforced concrete slab is dependent upon the arrangement of the bars provided that the spans and loads are the same. The bars should be arranged in two directions at right-angles to the directions of the greatest moments on the slab, the greatest amount of steel being placed where the largest deformations would occur in order to reduce these deformations, and the bars bent so as to follow the distribution of the stresses. The bending moments for which the steel is calculated are the greatest moment on the slab, the moment at a right-angle to the greatest moment, the moments along the edges of the slab, the torsional moments at the corners, and the moments

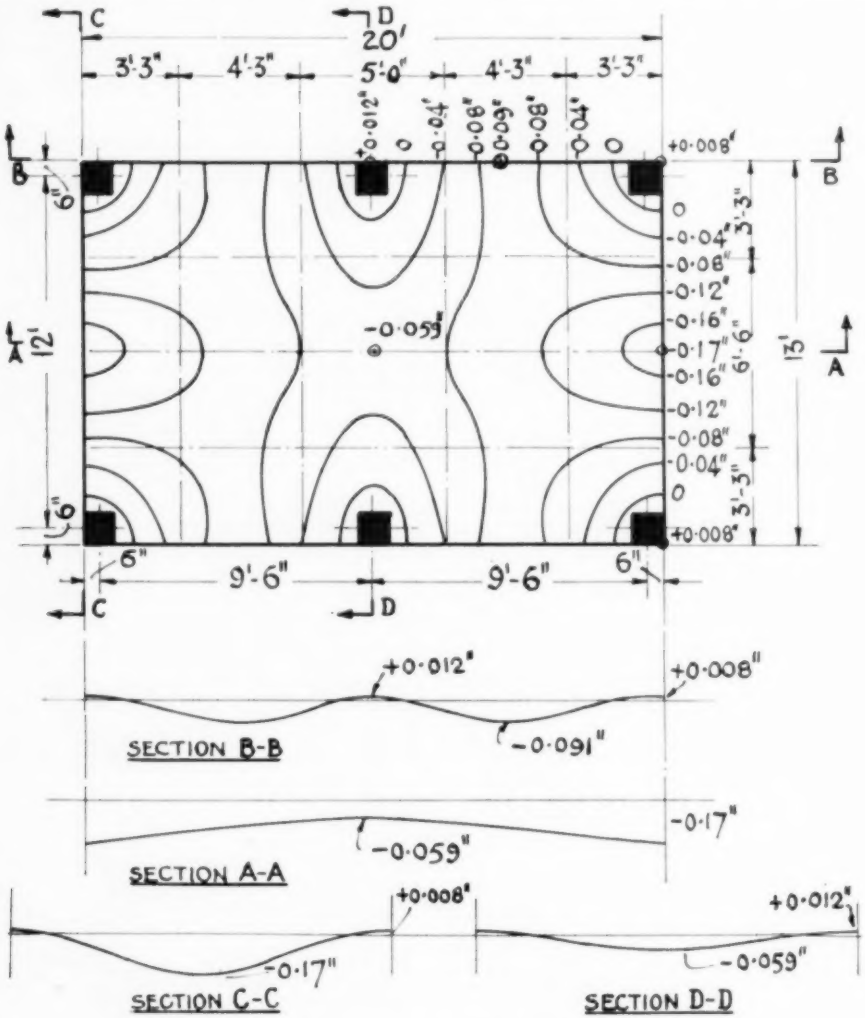


Fig. 2.—Deflections of a Slab 20 ft. by 13 ft.

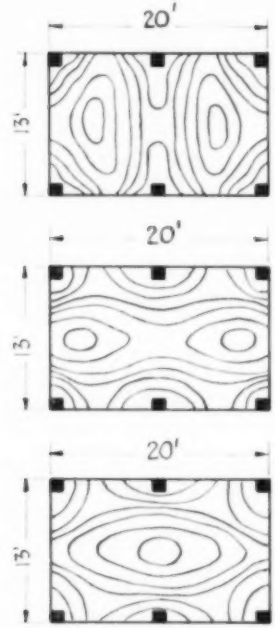


Fig. 3.—Deflections of Slabs with Different Arrangements of Bars.

at the supports. Other bending moments are not significant.

An example of a slab 6·1 m. by 4 m. by 10 cm. (20 ft. by 13 ft. 1½ in. by 4 in.) used as the ground floor of a house is shown in *Fig.* 1. The slab is supported on short columns at the corners and at the centres of the longer sides. By building the floor above the level of the ground (*Fig.* 5) and making a joint between the soffit of the slab and the top of the column it is possible to isolate the house from the effects of movements of the ground. The remainder of the structure is then supported by the slab, the walls being of concrete blocks about 8 in. thick and the roof of timber trusses covered with tiles. The total dead weight supported by the slab is 25 tons, that is 213 lb. per square foot, and the imposed load is about 50 lb. per square foot. The weight of reinforcement required is 333 lb. and the concrete contains 506 lb. of cement per cubic yard. The maximum compressive stress in the concrete is 710 lb. per square inch; the maximum tensile stress in the steel is 21,300 lb. per square inch. In *Fig.* 2 is a plan showing the contours of the deflected surface of a slab and sections showing the deflections caused by the loads previously given, and with the arrangement of bars shown in *Fig.* 1. The maximum deflection is 0·17 in. By varying the arrangement of the bars the deflected surfaces shown in *Fig.* 3 are obtained.

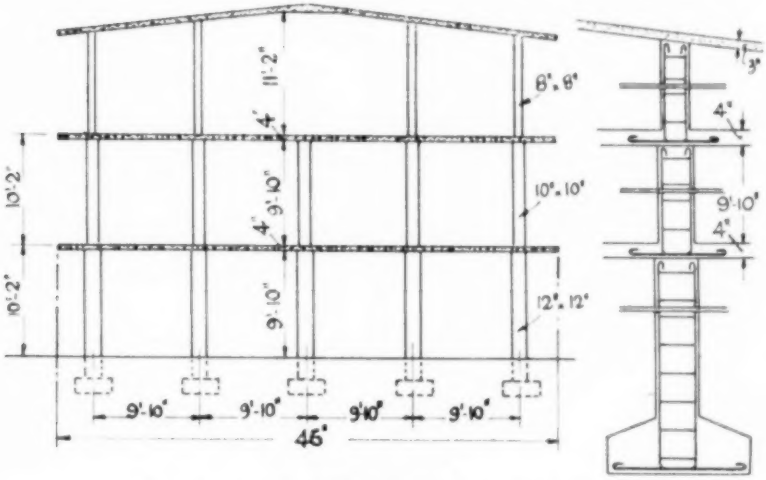


Fig. 4.—Cross Section through a Warehouse.

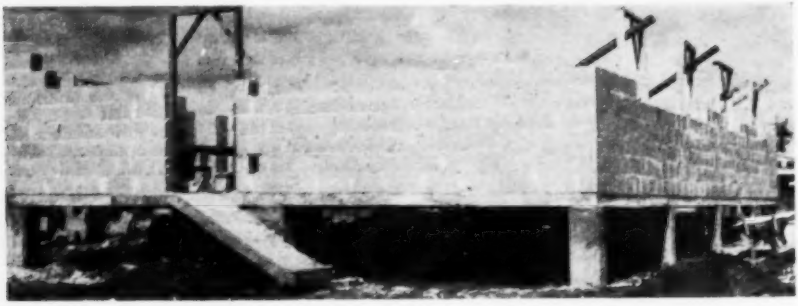


Fig. 5.—Slab for the Ground Floor of a House.

A cross section and details of the columns of a three-story warehouse near Haifa are shown in *Fig.* 4. The building is about 200 ft. long by 46 ft. wide, and the 4-in. slabs are supported at intervals of about 10 ft., in two directions, by five rows of columns with 21 columns in each row. The imposed load is 123 lb. per square foot and the weight of steel per square foot of slab is 0·92 lb.

We are informed that during the years 1949 to 1953 about 10,000 slabs similar to that described were erected in single-story and multiple-story buildings.

Casting Piles.

THE illustration shows the method used for casting piles at Alphen, Holland. From the mixer the concrete is transported in a skip mounted on a vehicle which takes it on to a bridge over the mould. The concrete is tipped on to the bridge, and is shovelled to the mould through a hole in the floor of the bridge. The bridge is carried on wheeled carriages at each side, and is pushed along the moulds as concreting proceeds.

A Blending and Bottling Warehouse.

THIS five-story building for the Distillers Agency, Ltd., at South Queensferry, which replaces buildings destroyed by fire in 1949, is of reinforced concrete with brick external and dividing walls. The main building (*Fig.* 1) accommodates the bonded stores, bottling hall, and case factory, with a loading-dock alongside new railway sidings extending the full length of one side. On the other side of the sidings are a new cooperage, cask store, garage, and receipt and despatch departments. The main building has an area of 5050 sq. yd.; the lower story is 18 ft. high and the upper stories 9 ft. 7 in.

A feature of bonded stores for the maturing of whisky in butts is that the fu" designed load on the floors remains undisturbed for periods up to seven years. Each floor carries two tiers of butts each weighing 12 cwt. It is not convenient to stack these butts higher, and the story height is unusually low in consequence. The live load specified for the bonded store was 3 cwt. per square foot. Floors used for other purposes were designed for a minimum live load of 2 cwt. per square foot owing to the possibility of future change in use and the installation of new plant. Part of the second floor supports

Fig. 1.

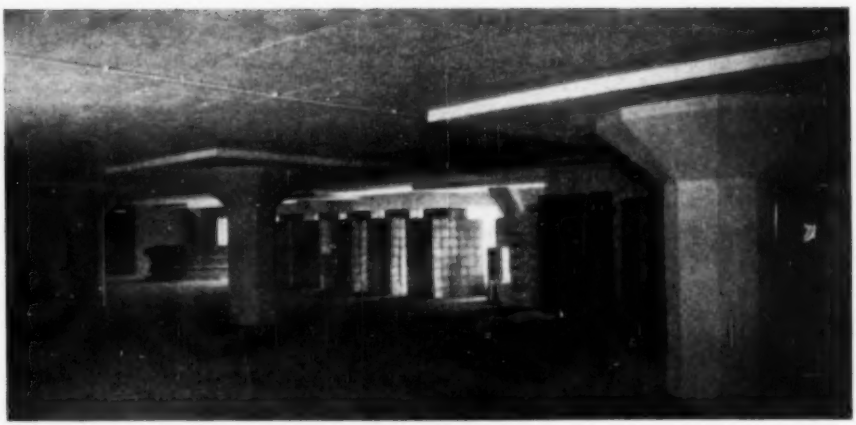


Fig. 2.

five circular blending vats, the largest being 18 ft. in diameter and weighing 100 tons, and they extend through the third floor. The floors in the main building are "flat slab" construction (*Fig.* 2).

The columns in the main building are spaced on a grid of 25 ft. by 24 ft. 2 in. The maximum load per column is 506 tons and this is transmitted to the ground through isolated bases at a maximum pressure of 2 tons per square foot.

The floors generally are 10 in. thick including a granolithic topping ¾ in. thick laid immediately after the concrete was placed. All reinforcement subject to tensile stresses was of high-tensile square steel bars.

A canopy curved in cross section over the loading dock extends the full length of 340 ft. of one side of the building, from which it projects 23 ft. 3 in.. The canopy is a reinforced concrete slab varying in thickness from 12 in. at the wall face to 3 in. at the outer edge and is cantilevered from the first-floor slab. The canopy is divided into three nearly equal parts separated by expansion joints. The top has been treated with two coats of waterproofing compound.

The flat roof is of similar construction to the floors. It is laid to falls by inclining the slab, and is lined with insulating board under the asphalt. No expansion joints are provided in the floors or roofs, but some joints have been formed in the brick parapet walls which are exposed on both sides.

The building was designed by The Distillers Co., Ltd., and erected by Messrs. Melville, Dundas & Whitson, Ltd. Considére Constructions, Ltd., were consultants for the reinforced concrete work.

Conference on Prestressed Concrete in Canada.

THE conference on prestressed concrete (mentioned in this journal for September last) will be held at the University of Toronto on January 28 and 29, 1954. The committee comprises Professor C. F. Morrison (University of Toronto), chairman; Mr. H. Fealdman (Hydro-Electric Power Commission of Ontario), secretary; Mr. J. A. Gow, Professor M. W. Huggins, and Professor T. R. Loudon (University of Toronto); Mr. R. H. Macdonald (Steel Company of Canada); Mr. M. MacMurray (Engineering Institute of Canada); Mr. E. P. Muntz (consulting engineer, Hydro-Electric Power Commission of Ontario); Mr. D. O. Robinson (Canada Cement Company); and Mr. R. B. Young (concrete consultant of the Hydro-Electric Power Commission of Ontario).

The papers to be read include The applicability of prestressed concrete to Canadian construction, economic aspects, and future possibilities, by Mr. R. F. Shaw (Vice-president of the Foundation Company of Canada); Strain-gauge tests of 40-ft. and 60-ft. beams, and observations on the use of these beams in an industrial building, by Mr. A. T. Waidelich (Vice-president and Manager of Research of the Austin Company of Cleveland, Ohio); Quality control of concrete and suggested specifications, by Mr. R. B. Young and Mr. J. N. Mustard (Hydro-Electric Power Commission of Ontario); Quality of steel for prestressing and suggested specifications, by Professor M. W. Huggins; Significant features of prestressed concrete, by Professor G. Magnel; Basic theoretical consideration of prestressed concrete, by Dr. T. O. Lazarides; Reports on prestressed concrete research, by Mr. A. L. Parme (Portland Cement Association, Chicago); and Prestressed arch bridges in Venezuela, by Mr. Robert Shama (Empresas Campenon Bernard de Venezuela).

Fire-resistance Tests of Building Materials.

THE revision of the British Standard entitled "Fire Tests on Building Materials and Structures" (No. 476: 1953. Price 4*s.* from the British Standards Institution) includes details of tests to determine the combustibility of materials, the surface-spread of flames, and the fire-resistance of structural members. Definitions of combustibility and of the grading periods of fire-resistance, and the classification of surfaces based on the test for the spreading of flames, are given. For load-bearing members the load to be applied during the test is the design load and not 1½ times the design load as in the previous standard. The water and impact tests of structural members have been deleted. The apparatus required for the tests is described.

A North-light Shell Roof.

THE reconstruction of the offices of Messrs. Swan, Hunter & Wigham Richardson, Ltd., at Wallsend-on-Tyne, includes administrative offices of steel-frame construction with reinforced concrete floors, and a drawing office. The drawing office (*Fig.* 1) is supported on columns from 12 ft. to 20 ft. high between the soffit of the floor and the sloping ground below. The area under the drawing office is occupied by the main road to the shipyard, a concourse, cycle racks, and small office buildings.

The overall dimensions of the drawing office are 173 ft. 7 in. by 82 ft. The floor is a flat slab 9½ in. thick supported on columns of 1 ft. 6 in. diameter. There are 32 columns in two groups of sixteen columns each, and between each group the space is about 12 ft. The columns in each group are spaced 25 ft. apart in both directions. Over the 12 ft. space the floor cantilevers in two equal spans of about 6 ft., and here an expansion joint is formed through the building. The size of the flared heads of the columns has been increased for architectural reasons beyond that required by the regulations. *Fig.* 2 shows the area under the drawing office and *Fig.* 3 is a cross section through the building.

The roof comprises three bays of north-light shells, each bay being divided into two equal lengths supported by two columns, about 75 ft. apart, and cantilevering about 6 ft. to the central expansion joint. The curved portion of the shell is 2¾ in. thick and the mullions, 6 in. by 6 in., are at 7 ft. 6 in. centres and carried on a valley-beam 1 ft. 6 in. deep. The cross section of the columns is 1 ft. 3 in. by 1 ft. The stiffening frames are 1 ft. wide and vary in depth from 2 ft. 2 in. to 1 ft. 3 in. Around the perimeter of the roof is a slab 6 in. thick, forming a footpath about 4 ft. wide, supported by brackets from the columns and edge-beam.

On the roof are 2 in. of vermiculite covered with three-ply bituminous felt. The soffit of the shells is plastered with

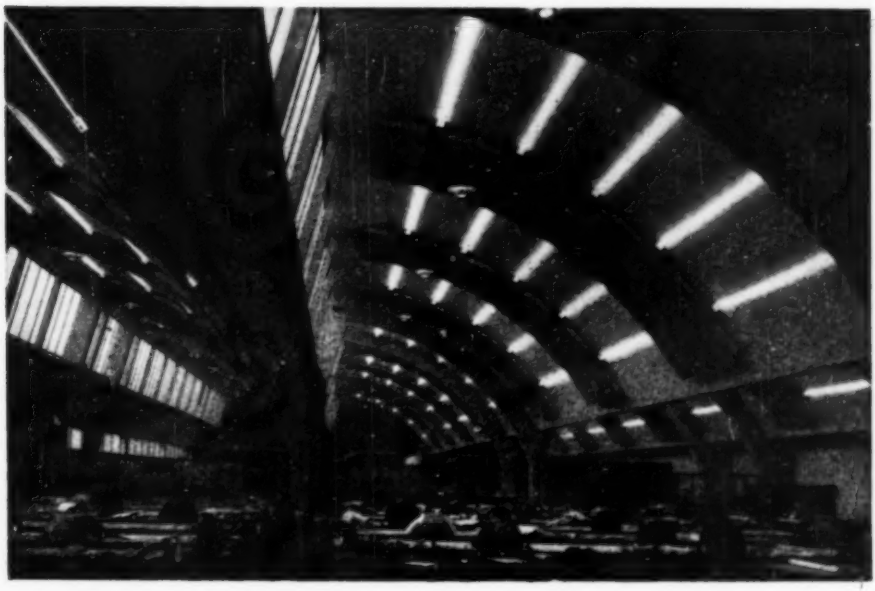


Fig. 1.—The Drawing Office.

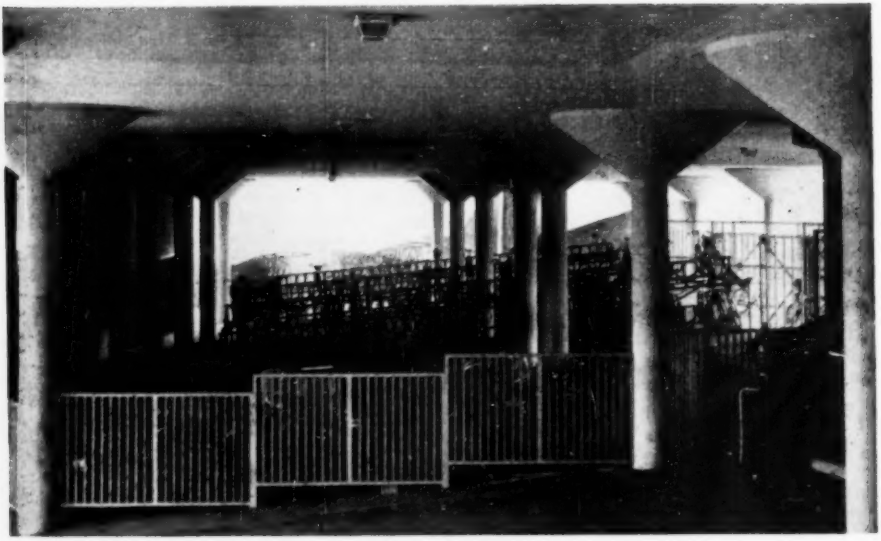


Fig. 2.—The Roadway under the Drawing Office.

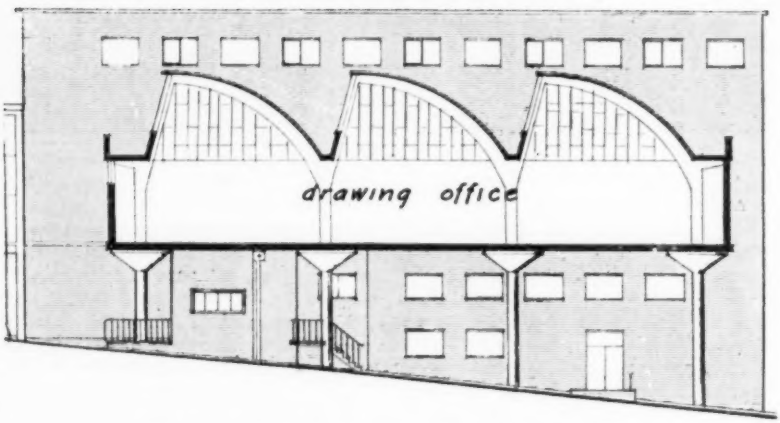


Fig. 3.—Cross Section through Drawing Office.

vermiculite, under which are acoustic panels measuring 21 ft. 6 in. by 6 ft. fixed at 15 ft. centres. The panels comprise perforated hardboard sheets attached to timber frames and suspended from the roof by mild-steel hangers. Rainwater pipes and electrical conduits are in the centres of the columns.

The expansion joint in the roof was formed by casting a nib 5 in. high along each edge of the adjoining shells. On top of the nibs were placed strips of timber and the joint was sealed with bituminous felt. The roofing felt was then carried over the joint and a strip of No. 21-gauge copper sheet fixed to the timber strips on one side only by copper nails the heads of which were brazed to the sheet. In the floor there is a heating duct where the expansion joint occurs, and the joint is sealed by bent copper sheet cast in the floor of the duct.

Timber shutters were used for the columns and the flared heads, and steel panels for the floor. Timber bearers were used between the steel panels at the mid-spans of the floors so that the props could be left undisturbed when the steel panels were removed. The curved portion of the shell was cast on steel sheets on a framework of scaffold tubes. The shuttering for the valley-beams spanned 25 ft. as it could be supported only on the columns under the floor.

The architects were Messrs. Richard Shepherd & Partners and the consulting engineers Messrs. T. F. Burns & Partners. The shell roof was designed by Mr. H. G. Cousins, B.Sc., M.I.C.E. The tender price, including a steel-framed office building five stories high and measuring 98 ft. by 21 ft., was £62,000, and the work was carried out by Messrs. Charles R. Price.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Prestressed Concrete, by J. S. Arlett. Marine and Technical College, South Shields. November 18. 7.15 p.m. By R. W. Pearson. 83–85 Graham Street, Airdrie. November 24. 7.15 p.m.

Concrete Placing and Formwork, by A. B. Harman. Technical College, Great Horton Road, Bradford. November 12. 7.15 p.m.; Schofield Technical College, Park Road, Mexborough. November 19. 7.15 p.m. By L. J. Murdock. Gas Showrooms, Town Hall Extension, Albert Square, Manchester. November 16. 7.15 p.m.; College of Technology, Byron Street, Liverpool. November 17. 7.15 p.m.

Basic Principles of Site Organisation, by H. E. Hewitt. Crown and Anchor Hotel, Ipswich. November 16. 7 p.m.

New Model By-laws, by R. A. Simons. Ministry of Works, Ashley Street, Birmingham. November 17. 7.15 p.m.

Thermal Insulation of Buildings, by C. S. Goyder. 26 Broad Street, Reading. November 17. 7 p.m. By R. R. Houston. Technical Institute, Southway, Bognor Regis. November 26. 7 p.m.

Soil Stabilisation, by S. MacKey. Technical College, Queen Street South, Huddersfield. November 17. 7.15 p.m.

Specialists on Building Contracts. North Staffs. Technical College, Cauldon Place, Stoke-on-Trent. November 18. 7.15 p.m.

Problems of Gypsum and Lime Plastering, by W. H. Ransom. Public Library, Corporation Street, Chesterfield. November 24. 7.15 p.m.

Soil Mechanics in the Building Industry, by M. W. Leonard. Therm House, St. Aldates, Oxford. November 24. 7 p.m.

Building Methods in the U.S.A., by W. R. Turner. College of Technology, Howard Street, Rotherham. November 24. 7.15 p.m.

Modern Developments in Concrete, by S. White. Technical Institute, Nelson Terrace, Stockton-on-Tees. November 26. 7 p.m.

Proposed Arch of 900 ft. Span.

IT is reported in the Portuguese press that plans are being prepared for a reinforced concrete bridge over the river Douro at Oporto with an arch span of 900 ft. The total length of the bridge would be 1600 ft.; the width would be 82 ft. An alternative design for a steel structure is also being prepared.

A Laboratory for Nuclear Research.

In this journal for March, 1952, a brief description was given of the nuclear physics research laboratory at the University of Liverpool to contain a 156-in. synchro-cyclotron weighing about 1700 tons. Further information regarding the construction of the building is given in the following.

The cyclotron room is 46 ft. wide by 60 ft. long by 17 ft. high, below which is a basement 8 ft. deep and a pit for the cyclotron a further 6 ft. deep. Above is an auxiliary-equipment room 52 ft. square by 25 ft. high, the greater width being obtained inside the room by building over the thick walls of the cyclotron room. Along one side of the cyclotron room and separated from it by a concrete wall 12 ft thick is an ante-room 52 ft. by 39 ft., above which are rooms for air-conditioning and other machinery.

The imposed loads for which the structure is designed are, in lb. per square foot: roof, 40; machinery-room floors, 224; other floors, 112. Allowance is made for exceptionally heavy equipment to be placed in certain positions. Lifting tackle can be attached to the ceilings at a number of points. Corbels on the walls of the cyclotron room support a 30-ton overhead travelling crane, and in the auxiliary equipment room a 10-ton crane. A gantry to lift 10 tons and a loading platform are on the west side of the building above an access road.

The whole of the external surfaces of the concrete is covered with asphalt.

Fig. 1.—The Walls of the Cyclotron Room.

Fig. 2.—Vertical Joint in Wall.

On vertical surfaces this was applied to a brick wall which acted as external shuttering (*Fig.* 1). The walls were built without heels to cheapen and simplify the asphalting. The lower part of the work and all foundations are in sandstone on which the bearing pressure does not exceed 4 tons per square foot.

Two of the walls of the cyclotron room are of plain concrete 6 ft. thick backed externally by pulverised sandstone filling taken from the excavation. The third wall is entirely below the existing ground

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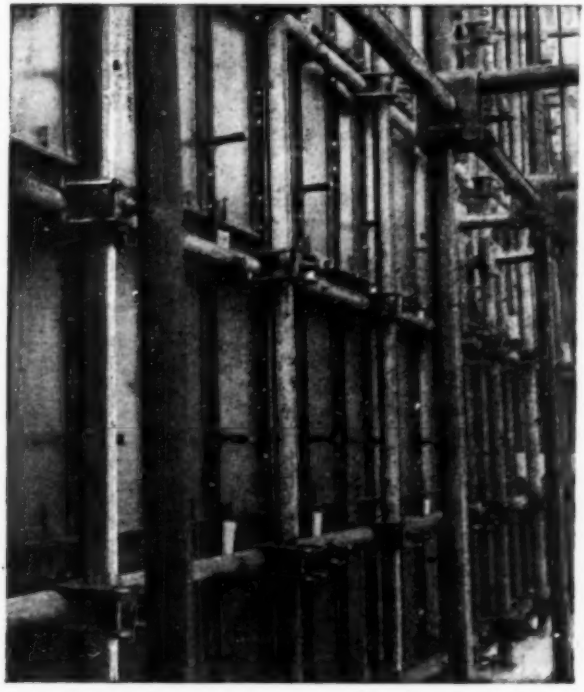

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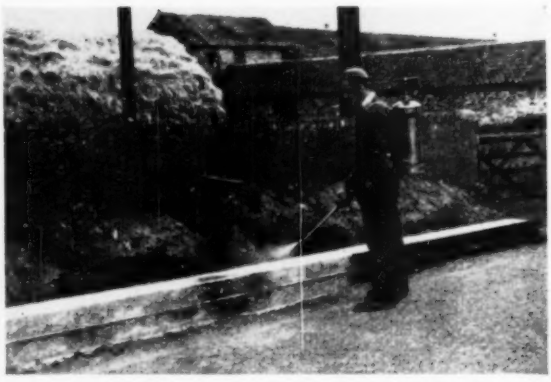

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level and is of reinforced concrete 2 ft. thick. The reinforcement in the lower part of this wall is shown in *Fig.* 1. The construction of the wall, 12 ft. thick, between the ante-room and the cyclotron room was delayed until parts of the cyclotron had been installed, and the floor above is designed to span independently of this wall. There are two openings in the wall. One is a doorway which can be sealed during experimental work by a sliding concrete " plug " 7 ft. diameter and weighing 25 tons. The other is a larger opening which is sealed with special blocks, described in the previous article, made of high-density concrete in which part of the coarse aggregate is replaced by steel punchings. Apart from the lintel over the large opening, the wall is unreinforced.

Above the cyclotron room, and forming the floor of the auxiliary-equipment room, is a reinforced concrete slab 4 ft. thick, covered by 1 ft. 6 in. of plain concrete in which are service ducts. The thickness of the floor required to give protection against radiation is sufficient to span the width of 40 ft. without beams. The numerous holes through the slab restricted the positions in which the reinforcement could be placed and made compressive reinforcement necessary in some places.

To reduce shrinkage cracks, coarse aggregate of a maximum size of 3 in. was used in the thicker members and sliding joints were provided to permit unrestrained movement as the concrete hardened. In the walls 6 ft. thick the vertical joints are about 22 ft. apart and consist of gaps 2 in. wide by 2 ft. deep on each side of the wall with a space 3 ft. by 2 ft. in the middle (*Fig.* 2). This central space was filled with concrete to form a key between the two parts of the wall. The joints in the slab 4 ft. thick are parallel to the span of the slab and on lines joining the vertical joints in the walls. They are formed by recesses 2 ft. wide at the top of the slab tapering to ¾ in. wide at the bottom. The sides of the slab are lined with bituminised hessian and filled with concrete. The V-shaped

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plugs so formed will tend to drop if further lateral contraction occurs and thus maintain contact with the sloping sides of the slab. The joints in the slab and the walls were filled with fine concrete, consolidated by vibration, at least six months after the walls and slab were cast ; it was estimated that by this time the greater part of the shrinkage would have taken place. To allow free contraction, the joints were continued through the walls and roof of the auxiliary-equipment room.

The mixture for the plain and reinforced concrete was 1 : 2 : 4 by volume. The crushing strength of cubes was required to be not less than 2850 lb. per square inch at an age of 28 days. The reinforcement consisted of electrically-welded hard drawn steel wire mesh and round mild steel bars.

In addition to the main building described there is a smaller building to contain a 37-in. cyclotron.

The architects were Professor W. G. Holford, M.A., and Mr. W. M. Shennan, in collaboration with Professor W. B. Skinner, F.R.S. The reinforced concrete was designed by the British Reinforced Concrete Engineering Co., Ltd., who also supplied and bent the reinforcement. The general contractors were Bovis, Ltd.

[An article giving the calculations for the thickness of concrete required to give protection against nuclear radiation at this laboratory is contained in the April, 1952, number of this journal.]

Unusual Method of Constructing a Tunnel.

THE illustration shows the method of constructing part of a tunnel for a railway in Stockholm. The tunnel is 2¼ miles long and is mostly in rock and here the usual methods were adopted. For the portions in earth the method shown in *Fig.* 1 was used in order that streets would be closed to traffic for a much shorter time than if excavation for the whole of the tunnel was carried out at once. The description and *Fig.* 1 are from " Engineering News-Record " for May 21, 1953.

The parts of the tunnel in earth are of reinforced concrete and the roof was cast directly on earth, the saving in shuttering partly compensating for the extra cost of the excavation. The first stage was to drive steel-beam piles clear of the foundation of the future tunnel. Earth was then excavated between the piles to the level of the soffit of the roof of the tunnel (*Fig.* 1*a*). Two inner lines of piles were then driven about 12 ft. from the first piles. The earth between the rows of piles on each side of the tunnel was then excavated (*b*) ; the piles were shored and sheeting was driven to form retaining walls as the excavation progressed. The walls and roof of the tunnel were then cast (*c*). Finally the earth inside the tunnel was excavated, the piles removed, and the road laid (*d*). To provide air-raid shelters at the stations and at the northern end of the tunnel a layer of rock 3 ft. thick was laid on the roof of the tunnel before repaving the street.

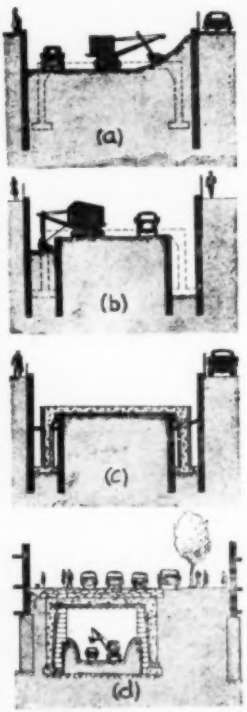
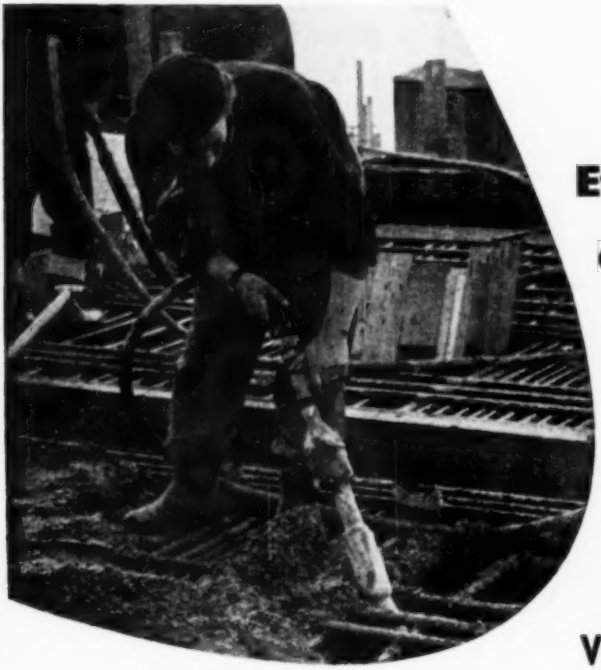


Fig. 1.



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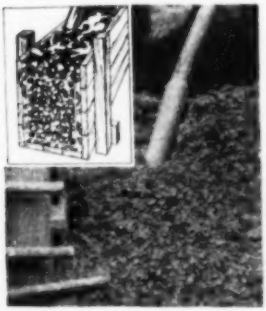

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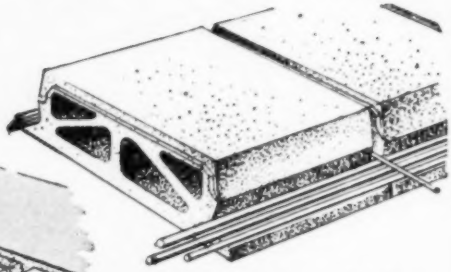

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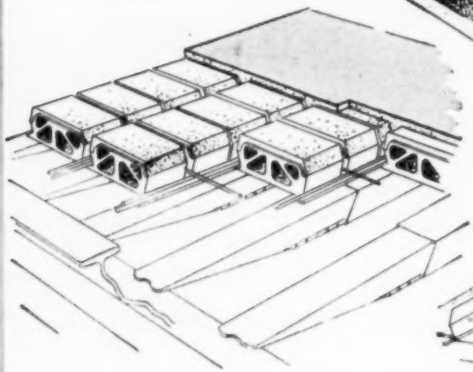

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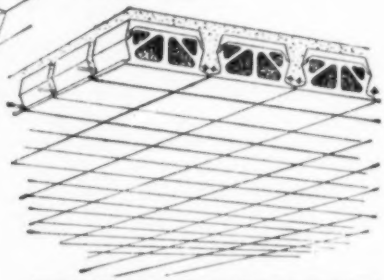

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Research on Concrete Roads.

THE following are abstracts from the report for the year 1952 of the Road Research Board (H.M. Stationery Office. Price 4*s*. 6*d*.).

Experimental Roads.

Plain and reinforced concrete slabs were laid between the years 1946 and 1947 on a road near Oxton, Nottinghamshire. The road was opened to traffic in March, 1948. Slabs from 4 in. to 8 in. thick were laid directly on a well-graded gravel-sand subgrade of uniform quality, and slabs from 3 in. to 6 in. thick were laid on various thicknesses of sand-cement base. The rate of development of new cracks is now very much slower than during the early life of the road. All the reinforced slabs are still in a satisfactory condition and, although the slabs 3-in., 4-in., and 5-in. thick have numerous fine cracks, the reinforcement is effectively holding the cracked portions together. The unreinforced slabs 7 in. and 8 in. thick in the nearside traffic line of the dual-carriageway road are still in a satisfactory condition. Unreinforced slabs 3 in. and 4 in. thick have already been replaced and many slabs 5 in. and 6 in. thick are badly cracked and, owing to the rapidity with which these cracks are spalling, will need replacement in the near future.

Experimental roads were laid during 1951 and 1952 to obtain information on the greatest length of reinforced slabs without joints that can be used without undue cracking, to determine the amount of reinforcement required to prevent cracking in slabs of various lengths, to compare the performance of slabs laid on flexible and rigid bases, and to compare dummy contraction joints with dummy warping joints between expansion joints. The dummy contraction joints comprise a groove $\frac{1}{2}$ in. wide by $2\frac{1}{2}$ in. deep formed at the top of the slab and without dowels or tie-bars. The reinforcement was stopped 2 in. on each side the joint. The dummy warping joints act as contraction joints but with a strip of mesh reinforcement 3 ft. 6 in. wide placed across the joints 4 in. below the surface.

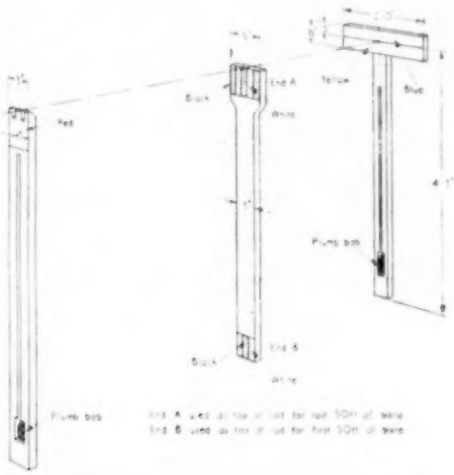

Fig. 1.—Boning Rods for Levelling Road Forms.

Levelling Side Forms.

Tests have shown that when the side-forms are levelled with boning rods errors up to $\frac{3}{4}$ in. may occur. Two more accurate methods have been developed, one employing specially designed boning rods (*Fig.* 1) and the other using a

stretched line or wire. With the new rods, movement of the line of sight from the true line is immediately noticeable. A suitable high-tensile steel wire for the second method is No. 20 or 22 S.W.G. pulled with a force of 200 lb. or 150 lb. at the ends of a length of 100 ft. An allowance for sagging of the wire may be made by means of a small jig. The best method of supporting the side forms during levelling is to support one end on the connecting bar of the adjacent form and the other end by a stake. As soon as the length is level the forms should be packed with cement mortar. When it is not possible to do this within a short time, blocks and shims should be used instead of stakes to support the forms during alignment.

Uniformity of Spreading Concrete.

Two methods have been developed for measuring the uniformity of spreading concrete and assessing the amount of surcharge required (the surcharge is the height of the freshly-placed concrete above the forms). One method consists of cutting 6-in. diameter cores from the freshly-placed uncompacted concrete to determine the variation in density from place to place. Another method uses the work done to obtain a given penetration of a steel plunger, 3 in. diameter, into the freshly-placed concrete. The first method is particularly suitable for determining the surcharge required to obtain full compaction of an unreinforced slab when the surface is compacted to the level of the forms. Since the test is made on the fresh concrete it can be used at the beginning of new work to ascertain the correct surcharge and thus to ensure that the concrete is suitable for the plant available. The second method is more convenient when it is desired to assess the uniformity of the spread concrete.

Non-destructive Testing.

Experiments show that the only factor seriously affecting the use of the longitudinal wave velocity as a criterion of the quality of concrete is the ratio of aggregate to cement. The type of aggregate and the moisture content affect the relation between the longitudinal wave velocity and compressive strength but do not introduce discrepancies with respect to the modulus of rupture or density. Calcium chloride does not affect the longitudinal wave velocity; it increases the compressive strength and reduces the modulus of rupture. Tests on slabs containing bars between 0·375 in. and 1 in. diameter, using transducers of 1 in. diameter, showed that, when the pulse was transmitted perpendicularly to the axis of the bar, appreciable change of velocity occurred only when the diameter of the bar approached that of the transducer. When the waves were propagated in a direction parallel to the axis of the bars, two separate signals were received when the bars had diameters exceeding 0·6 in., of which the initial signal through the steel was smaller than that through the concrete. Only a small amount of energy was coupled into the bars, and the signals could be distinguished. Since the signal in the bar precedes that in the concrete, care should be taken to avoid propagating the pulse near the axes of thick bars.

Joints with Dowel-bars.

The value of dowel-bars is shown by some tests that have been carried out on a heavily-loaded part of the Chertsey trunk road. This road was constructed in 1930 by the Ministry of Transport and includes transverse contraction joints of the plain butt, dowel-bar, tongue-and-groove, and sleeper types. The movements of these joints during the passage of a 7-ton lorry were measured during a night in June 1952, when the slabs were warped upwards at the edges and corners by the high temperature. The result showed that the relative vertical movement of all the dowel-bar joints and 13 out of 15 of the tongue-and-groove joints was less than 0·001 in. but that larger movements, in some cases up to 0·05 in., occurred in the plain butt and sleeper joints.

Machines for Re-sealing Joints.

The re-sealing of joints may be done quicker by the use of the following machines: (1) A rotary pick for removing debris wedged in old joints; (2) A rotary wire-brush for removing loose dust; (3) A pressure-sprayer for applying a tack-coat; and (4) A machine for pouring hot sealing compound. Five men using these machines can re-seal 10,000 ft. of joint

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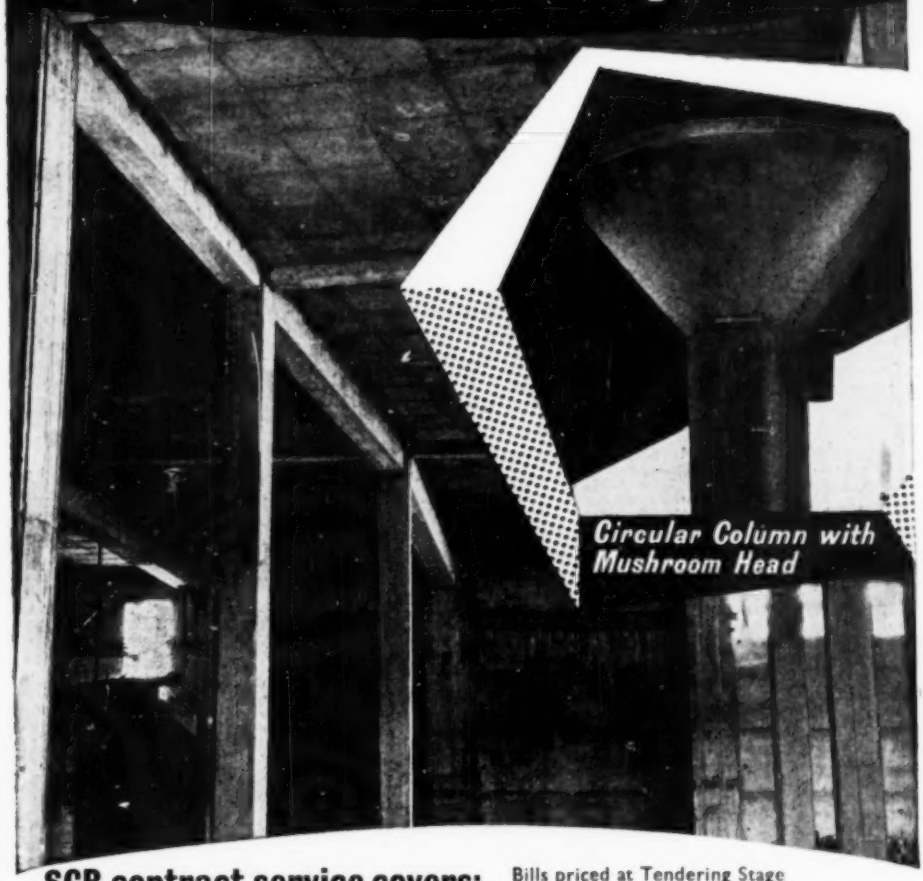

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Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. Reinforced concrete engineers require designer-draughtsmen, preferably A.M.I.Struct.E., for work at Bristol or near London. Opportunity to widen experience. Good salary to right men. Box 3648, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Consulting structural engineer, Westminster, requires senior designer-draughtsman with first-class experience in reinforced concrete for responsible position. Experience in structural steelwork an advantage. High salary and good prospects for suitable applicant. Write in confidence, stating age, qualifications, and full details of experience. Box 3685, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Experienced reinforced concrete designer required, age 25 to 30, for expanding drawing office. Work involves wide range of structures. Pension scheme. Write, stating age, experience, qualifications, and salary required. Box 3687, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Qualified geologist required with some experience of soil mechanics technique for Midlands office of firm of structural engineers. Apply, stating age, experience, qualifications, and salary required. Box 349, 19/21 Corporation Street, Birmingham, 2.

SITUATIONS VACANT. Twisteel Reinforcement, Ltd., have vacancies for designers and detailers in their London office. Apply, stating experience and salary required, to 43 Upper Grosvenor Street, London, W.1.

SITUATION VACANT. Chief assistant required to chief engineer of well-known London firm of designer suppliers. Salary according to experience. Write, stating experience, to Box 3689, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. YORKSHIRE ELECTRICITY BOARD. Applications are invited for the following appointments: Area Board Headquarters: Senior Draughtsman (Civil)—Applicants must have good experience in preparing detailed drawings of reinforced concrete and general civil engineering work and be able to work with the minimum of supervision. Salary, N.J.B. Schedule D, Grade 8, £567–671 per annum. Engineering Draughtsman (Civil)—Applicants should have experience in preparing detail drawings of reinforced concrete work, structural steelwork and general civil engineering work. Salary, N.J.B. Schedule D, Grade 6, £433–567 per annum. Applications, giving full details of age, qualifications, and experience, together with the names of two referees, should be forwarded to the SECRETARY, YORKSHIRE ELECTRICITY BOARD, Wetherby Road, Scarcroft, within fourteen days of the appearance of this advertisement.

SITUATION VACANT. Experienced reinforced concrete designer required for work in the project section of a company of reinforced concrete engineers and contractors. Five-days' week and pension scheme. Write, giving full details of age, experience, and qualifications, to Box C.E. 578, c/o 191 Gresham House, London, E.C.2.

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SITUATIONS VACANT. Designers. SIMON-CARVES LTD. have two additional vacancies for reinforced concrete designers. At least 10 years' experience of heavy industrial structures is essential. A good technical level, preferably Associate Membership, will be expected. Salary according to experience, but not less than £750. Excellent working conditions. A pension scheme is in operation. Apply, giving age and experience, to STAFF & TRAINING DIVISION (Ref. S.J.17), SIMON-CARVES LTD., Birdhall Lane, Cheadle Heath, Cheshire.

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SITUATIONS VACANT. THE BRITISH REINFORCED CONCRETE ENGINEERING CO. LTD. have vacancies for detailers and designers within a salary range of £500 to £1,200 per annum, according to experience, in their Stafford and London offices. These appointments offer wide experience in the design of reinforced concrete, prestressed concrete and shell structures. Staff pension scheme, five-days' week, and staff canteens. There are also vacancies in Glasgow and Newcastle-upon-Tyne.

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(*Continued on page 370.*)

in a normal working day. Machines of all four types are illustrated in the report.

A Method of Sealing Joints Temporarily.

During the development of a housing estate the roads are usually built first and are often covered with mud and stones. If the joints are sealed with ordinary compounds they are unable to withstand these conditions. Strips of soft-wood, with a tapered cross section, wedged into the joints on top of a layer of soft bitumen are stated to provide an effective seal and completely resist the ingress of grit for a period up to two years. Afterwards the wood can be removed and the joints sealed in the normal manner.

A Jack for Tensioning Single Wires.

A JACK weighing 56 lb. and which can be operated by one man is shown in *Fig.* 1. It may be used to tension single high-tensile wires up to 0·276 in. diameter and with a total extension of 4½ in.; similar jacks give extensions up to 1 ft. 2 in. The jack is suitable for pre-tensioning and post-tensioning methods of pre-stressing concrete.

The pump and jack are in the centre of the machine (*Fig.* 1) to the rear of which is a manometer. The rear of the jack presses against two angle-sections over which is a small reservoir for oil. The angles are separated by two arms, loosely bolted between the angles and extending towards the front of the jack. Projections at the ends of the arms fit into slots formed in a tube, against one end of which is placed the ram of the jack. The other end of the tube is shaped to fit between the cylindrical anchors for the wires.

To tension a wire, the cylinder of an anchor of the type shown in *Fig. 2a* is threaded over the end of a wire projecting from the anchor-plate and two small serrated wedges are placed loosely in the cylinder and around the wire. A second anchor, of the type shown in *Fig. 2b*, is then placed a certain distance away from the first anchor, the distance being determined by a gauge. The wedges in this anchor are spring-loaded so that a wire may be pulled through the anchor in one

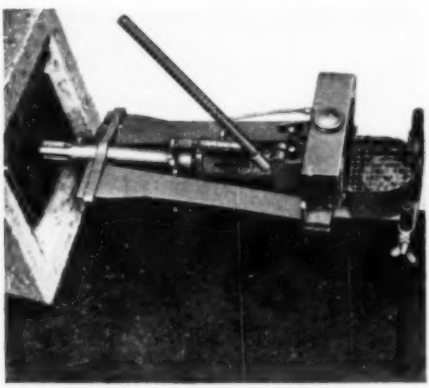

Fig. 1.—The Jack.

Fig. 2a.—A Standard Anchor.

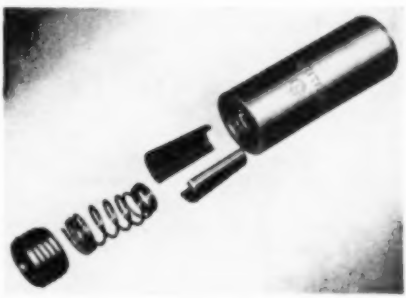

Fig. 2b.—A Spring-loaded Anchor.

direction but a pull in the opposite direction tightens the wedges and anchors the wire. The tube is then slid over the wire so that the shaped end bears against the anchor-plate. The jack is placed against the other end of the tube and the arms are fitted into the slots on each side of the tube, being retained in position by the movable clamp shown in *Fig.* 1 extending across both arms. The ends of the arms projecting into the slots are placed between the two anchors so that when the rear of the jack is moved away from the anchor-plate the two arms, engaging the front of the spring-loaded anchor, extend the wire; the reaction is provided by the tube pressing against the anchor-plate embedded in the member. The extension may be measured on a rule fitted on top of the jack, and the force is indicated on the manometer. When the desired extension is obtained the wedges in the anchor bearing against the anchor-plate are driven in by a tool inserted in a small slot in the top of the tube and immediately behind the anchor. During the tensioning operation these wedges are kept in position by a metal insert placed in this slot. When the wire is anchored the arms are disengaged from the slots and the jack removed. The projecting wire is then cut and the spring-loaded anchor slid off the short length of wire. It is claimed that 20 wires per hour may be tensioned by this means.

The jacks and anchors are designed for a method of prestressing known as the Gifford-Udall-CCL system in which cables comprising up to twelve 0·276-in. wires are used. A cable of eight wires requires a cavity 1½ in. diameter in the member and a cable with 12 wires a 2-in. cavity. At the ends of a member the cavities are increased in diameter over a length of about 9 in. to allow the wires to pass through holes at $\frac{15}{16}$ in. centres in the mild steel anchor-plates. The jack was designed by Mr. E. W. Gifford in conjunction with Messrs. J. J. Udalls Building Co., Ltd., and Cable Covers, Ltd.

New Type of Batching Plant.

THE plant shown is in use for the construction of a bridge in Holland. The materials are supplied by a power-loader on a tractor to the hoppers, which have a total capacity of about 3 tons. The weighing skip under the hoppers slides on rails, and can be discharged to a mixer on each side.

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(Concluded from page lx.)

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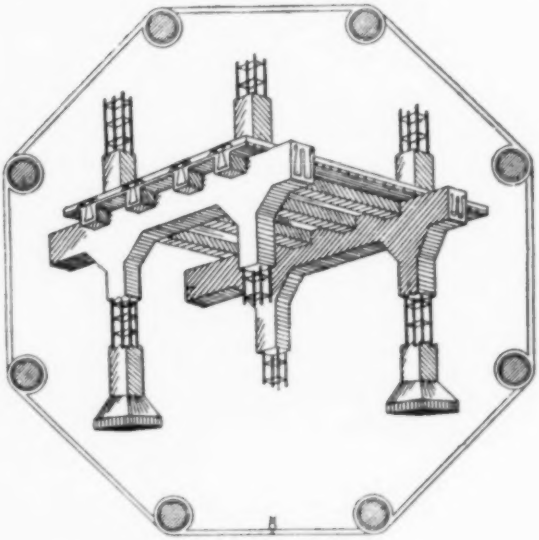

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